

AN EVALUATION OF DESIGN  
HIGHWATER CLEARANCES FOR PAVEMENTS

By

MOHAMED K. ELFINO

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DEDICATED  
TO THE MEMORY OF MY MOTHER, WHOSE  
SILENT PRESENCE HAS GUIDED MY EFFORT  
  
TO MY LOVING WIFE NARIMAN AND MY  
DAUGHTER NANCY FOR THEIR SUPPORT AND LOVE

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Mohamed K. Elfino

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Chairman: Dr. John L. Davidson  
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This research was carried out to investigate the effect of water presence on the permanent deformation of four subgrade soils. First, the physical and engineering properties were determined, with emphasis on developing the soil-water retention characteristic of the four soils. Second, repetitive load testing was performed at two different water conditions. These conditions were (1) at optimum, to represent the as-built condition, and (2) at varied water retention conditions to represent the subgrade condition in-service, i.e. in equilibrium with the designated water table. The deformation characteristics of the subgrade fill, at different water conditions, were related to pavement rutting in accordance with the Shell Criteria. When a tolerable deformation was obtained from a specimen at a specific water condition, its location on the soil-water retention curve was determined and the height of the subgrade fill fixed.

Based both on the height of the capillary fringe and on the Shell Criteria, the following subgrade fill heights were found to be adequate; 24 inches for the A-3 soil, 36 inches for the A-2-4 soil, and 12 inches for the A-2-6 soil. If the A-5 soil must be used, further careful analysis should be undertaken. A fill thickness in the range of 4 to 6 feet is probably adequate.

Because of the possible variations in soil within any classification grouping, the results of this research are limited to the particular soils investigated.

## CHAPTER I INTRODUCTION

### 1.1 Background

The presence of water in highway pavement systems accelerates the deterioration and destruction of the pavement. This has long been recognized. Great road builders of the past, such as Pierre M. Tresquet of France and John L. McAdam of Great Britain, knew of the importance of keeping roadbeds dry and protecting them against water damage.

In 1820, John McAdam stated:

The roads can never be rendered thus perfectly secure until the following principles be fully understood, admitted and acted upon: namely, that it is the native soil which really supports the weight of the traffic; that whilst it is preserved in dry state it will carry any weight without sinking . . . that if water pass through a road and fill the native soil, the road whatever may be its thickness, loses support and goes to pieces.

Pavement systems must be designed such that either water is prevented from entering places where it can cause damage, or alternatively, any water which does enter can be quickly and safely removed.

Figure 1.1 shows six ways in which the moisture content of a highway subgrade can be changed. These are

1. Downward flow through joints, cracks, and porous surfaces.
2. Lateral flow from water ponded on high medians.
3. Upward flow from high groundwater, springs, or rivers.

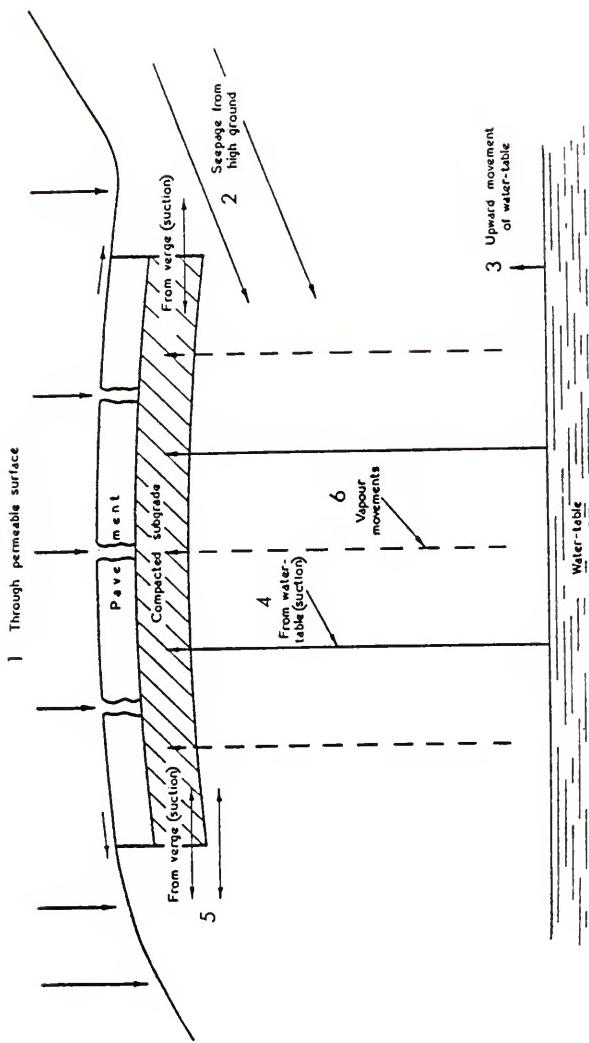


Figure 1.1 Ways in which Water Can Enter a Highway Subgrade (from Road Research Laboratory, 1955)

4. Capillary suction from underlying water table.
5. Transfer of moisture either to or from the soil in the verges as a result of differences in moisture content.
6. Condensation of water vapor as a result of fluctuations in temperature and other atmospheric conditions.

The research described in this dissertation relates specifically to the fourth cause listed above, capillary suction. To prevent water rise into critical areas of the pavement system or to reduce its effects, some State Departments of Transportation have developed highwater clearance guidelines. In these guidelines a minimum height, the clearance, between a groundwater level and a particular elevation within the pavement system is specified.

The Florida Department of Transportation (FDOT) has such guidelines. They state that the bottom of the pavement base should be located a specific height above a designated highwater level. The details are as follows:

4-lane Primary and Interstate	3 feet above 50-year highwater
2-lane Primary	2 feet above 50 year highwater
Secondary Roads	1 foot above 25 year highwater

The policy has been applied uniformly statewide, and is intended to satisfy two concerns:

1. Achieving the required compaction and stability during construction operations.
2. That adequate pavement performance is provided.

The guidelines, however, do not address a most critical factor, that of subgrade soil type. As a result, there have been locations where the specified clearance appeared excessive for the conditions encountered. However, with no other guidelines to follow, the policy

was adhered to. The resulting high fill costs may not be justified. At the other extreme, situations may exist where the specified clearance may be inadequate. This also leads to eventual waste of funds due to construction problems or poor pavement performance.

To take subgrade soil type into account in setting such guidelines requires a determination of which physical and mechanical soil properties influence behavior during construction and under the expected dynamic service loadings. It is also very important that the water present in the subgrade be correctly modeled in any laboratory testing, i.e., correct water content and water pressure. This research evolved from a recognition of the need to include subgrade soil type in high-water clearance guidelines and the problems involved in doing this.

### 1.2 Purpose of Study

The purpose of the study was to investigate the soil-water retention behavior of four different Florida soils, to determine their deformation characteristics under repetitive loading at different water retention conditions, and to evaluate the suitability of the soils as subgrade material for a major highway.

### 1.3 Scope of Work

This study was planned in six phases, as follows:

#### First Phase

This phase consisted of two parts. First, a questionnaire survey was sent to all State Departments of Transportation to ascertain if they used highwater clearance specifications and, if so, what were the bases for their criteria. A literature review was then made in the areas of soil water retention in subgrades and of repeated load testing.

#### Second Phase

The physical and engineering properties were determined for four different materials commonly used as fill by FDOT. These properties included grain size distribution, Atterberg limits, specific gravity, permeability, soil mineralogy, soil-water retention characteristic, density-water content relationships, cohesion, and internal friction angle.

#### Third Phase

This phase consisted of column studies to simulate field conditions for compacted and stabilized subgrade in order to obtain the soil-water characteristic curves.

#### Fourth Phase

The deformation characteristics of the four subgrade materials compacted at optimum water content were determined. This represents the pavement condition as built.

#### Fifth Phase

The deformation characteristics of the four subgrade materials at varied soil-water retention conditions were determined. This represents the subgrade condition in-service, i.e., in equilibrium with the designated water table. Repetitive axial loading was used in both the fourth and fifth phases.

#### Sixth Phase

The deformation characteristics of the subgrade fill, at different water conditions, were related to pavement rutting potential in accordance with the Shell Criteria. When a tolerable deformation was obtained from a specimen at a specific water condition, its location on the soil-water retention curve was determined and the height of the most economical subgrade fill then fixed.

CHAPTER II  
REVIEW OF LITERATURE

2.1 Water Retention in Subgrades

In 1897 Briggs developed a classification describing soil water in the following manner:

- a. Hygroscopic water which is held in thin films on the soil particles by adsorption.
- b. Capillary water which is held in soil voids by surface tension.
- c. Gravitational water which moves in and through the voids under the influence of gravity and which drains from the soil when it is held at a higher elevation than the water table.

Soil moisture content is commonly expressed as the percentage of water weight per unit weight of oven dry soil. Experience has shown that moisture percentages are of little significance in predicting soil behavior, unless something is known of the texture and mineralogical nature of the soil itself. A more reliable approach in characterizing soil moisture conditions is to base it on the security or tenacity with which the water is held by the soil.

In 1907, Buckingham introduced the energy concept which provided a means of describing soil moisture conditions in terms of a potential function which is a property of the state of stress of the soil water itself. Buckingham reasoned that the flow of moisture through unsaturated soils was similar in many respects to the flow of electricity through a wire or the flow of heat through a rod. The flow

in each case being from regions of high potential to regions of lower potential. In the case of moist soil in which the capillary potentials are balanced with the gravitational potential, the capillary soil moisture will approach a state of static equilibrium with a water table below and no upward or downward flow will take place.

For texturally homogenous soils in which the moisture is in equilibrium, the soil will be saturated at the water table and will decrease in moisture content with distance above the water table.

In 1920 Gardner, making use of Buckingham's energy concept, stated that capillary potential was equivalent to the pressure potential in Bernouilli's equation, and consequently could be determined by measuring the soil water pressure. He also noted that the capillary potential was equal to the negative pressure (tension). In 1941, Russell and Spangler brought to the attention of highway engineers the Buckingham capillary potential relationships in soil moisture. They believed this concept would be a useful tool in the study of subgrade moisture. In 1944, Kersten reported on such a study. He investigated the field moisture of the upper six inches of the subgrade near the interior of flexible pavement systems. His work is summarized as follows:

- a. The degree of saturation existing in the subgrades of numerous projects in 6 states averaged 73 percent, the range being from 60 to 81 percent.
- b. The subgrade soils of projects in which a high average percent of saturation occurred were in most instances either clay or silty clay.
- c. Saturation percentages varied with soil texture; in general, they were high for the clays and became progressively lower for the clay loams, the loams, and the sandy loams.

*Subgrade*  
d. Soils of groups A-6 and A-7 showed higher average percentages of saturation than those of groups A-1, A-2, and A-4.

e. Subgrade moistures expressed as percentages of the plastic limit for a large variety of soils in six states averaged 77 percent.

Averages for individual states varied from 64 to 82 percent.

Approximately 17 percent of the determinations disclosed moisture contents in excess of the plastic limit.

f. The fine-textured soils, such as clays, exhibited a marked tendency to attain moisture contents in excess of their plastic limit. Sandy loams rarely had moisture contents as great as their plastic limit. Loessial silty soils tended to attain moistures close to their plastic limit.

g. The optimum moisture contents of the soils were exceeded by the field moistures in about one-third of the determinations reported. Clay soils exceeded the optimum most commonly, but soils of all textures, including the sandy loams, had moistures greater than optimum in a substantial proportion of the tests.

h. Only slight changes in moisture content for periods of from 1 to 5 years were indicated in tests on several projects. Soils on most of these projects, at the time of the initial tests, already had moisture contents approaching the plastic limits.

i. Clay soils with high percentages of saturation were encountered in areas with annual precipitations as low as 14 inches. Most tests in such regions, however, give relatively low saturation values.

The presence of water in a pavement system may decrease its strength in several ways, as summarized by Barber and Sawyer (1952):

1. It reduces the cohesion by lowering the capillary forces.

2. It reduces the friction by reducing the effective weight of the material below the water table.
3. For quickly applied loads, it may reduce the strength by the development of excess pore pressure.
4. Bearing capacity of sand is decreased more than 50 percent due to complete submergence as compared to dry sand. Capillary saturation gives somewhat less reduction.
5. For loose saturated sand under dynamic loading, the tendency to become denser causes pore pressures to increase. This reduces the effective normal stresses and thereby reduces the strength.
6. While high permeabilities may be obtained by using coarse aggregates, care must be taken to prevent a reduction in their permeability and stability by the intrusion of adjacent finer soils.

Darter et al. (1982) gave a list of several of the more common distress types, for flexible and rigid pavements, which are caused by water presence in the pavement system. The list included surface deformation in the form of distortion, corrugation, rutting, waves, depression, and potholes for the flexible pavement, while for rigid pavement the surface deformation is in the form of pumping and faulting.

Roque and Ruth (1983) in their analysis of pavement cracking on the Florida Turnpike, found that the in situ water content of the embankment soil of the uncracked pavement was six percent less than the cracked pavement sections. This increase in water content would substantially reduce foundation strength, making it susceptible to localized shear failure or high deformation under stress. (Haynes and Yoder (1963) found that the degree of saturation has substantial effect on the repeated

for over 100 years

load deformation properties of the AASHTO Road Test crushed stone and gravel materials. In these tests, the total deformation after 1,000 load cycles increased markedly once the moulding saturation exceeded about 85 percent. In contrast with this, the total deformation of specimens at saturation between 70 and 85 percent showed only a slight dependence on the moulding saturation.

Monismith and Finn (1977) reported that water presence in the pavement system is one of the most important environmental factors, particularly because it influences the response of the materials in the pavement section to load, and because it may cause undesirable volume change. They also stated that for design purposes, the influence of water may be considered by measuring the properties of materials at water contents, which are assumed representative of those that may develop at some time subsequent to construction.

Chu et al. (1977) indicated that the results of investigations on the variation of subgrade moisture, which were conducted in the United States and abroad, showed that, after a certain period of time following construction, subgrade soils below impervious pavements remain at a fairly stable moisture condition, except for a zone close to the pavement edge. Under idealized conditions and in areas where frost penetration does not extend to the subgrade, the variation in subgrade moisture depends mostly on the relative elevation of the groundwater table if it is within a certain depth below the pavement. In this respect, the critical depth of the groundwater table is dependent primarily upon soil type. Russam (1970) summarized the finding from the previous investigations and stated that, for highway and airport pavements, this depth would be approximately 20 feet in clays, 10 feet

in sandy clays or silts, and 3 feet in sands.) The moisture content of subgrade soils may then be estimated on the basis of the depth of the water table below the pavement together with soil suction data, provided that a moisture equilibrium condition has been reached in the subgrade. It could then be concluded from the previous studies that subgrade soils below pavements are seldom saturated, as is often assumed in formulating laboratory test procedures for the evaluation of subgrade soils. For this reason, the common practice of soaking soil specimens in water for a number of days before using them for laboratory tests, such as in the California bearing ratio (CBR) test, may result in an extremely severe moisture condition which usually does not occur under the pavement. If pavement structures are to be designed by rational procedures, simulation of anticipated field moisture is necessary. Burland (1965), and Aitchison and Richards (1965) emphasized the need of simulating negative pore pressure and utilizing equilibrium suction in determining the behavior characteristics of partially saturated soils.

Christensen (1940) reported that if a column of soil is in equilibrium with a water table and there is no tendency for the water in the soil to move either upward or downward, then for each foot of increase in elevation above the water table the tension in the water should increase by an amount equal to one foot of water. This relation between the tension and the elevation in the column can be represented by a straight-line extension upward from the water table with unit slope ( $45^\circ$  for the scale being the same for the abscissa and the ordinate). Similar conclusions, to those of Christensen, were reported by Croney et al. (1958).

Capillarity in soils is similar in many respects to the rise and retention of water in a capillary tube, although there are also important differences between the two cases. It is convenient to introduce the subject of capillary water in soil by reviewing the action of water in a capillary tube. It is well known that if a clean glass tube having a fine bore is placed vertically in a container of water, the water in the tube rises above the level in the container. Two phenomena are responsible for this rise:

- a. Forces of attraction between water molecules which at an air-water interface give rise to surface tension.
- b. Attraction of water to the material of the tube which causes wetting.

Capillary rise is related to surface tension, radius of capillary tube, and angle of contact as follows:

$$h_c = \frac{2T_s}{r\gamma_w} \cos \theta$$

in which

$h_c$  = height of capillary rise

$r$  = radius of circular capillary tube

$T_s$  = surface tension of water

$\theta$  = contact angle between the surface of the liquid and the surface of the tube

$\gamma_w$  = unit weight of water

Some typical values of capillary rise are given in Table 2.1.

Capillary water is at a pressure less than atmospheric, which creates capillary tension in the pore water and a counteracting effective stress known as capillary pressure. The effect of capillary stress is evident in the shearing strength of the damp, fine sand. The

Table 2.1 Typical Values of Height of Capillary Rise

Soil Type	Height of capillary rise $h_c$ , cm
Coarse sand	2-5
Sand	12-35
Fine Sand	35-70
Silt	70-150
Clay	200-400 and greater

Source: A. I. Silin-Bekchurin (1958)

wheel-load stability of damp sand on some beaches reportedly results from such pressures. When the sand is completely submerged or completely dry, this stability is largely lost.

Spangler and Handy (1982) stated that capillary water may be expressed quantitatively by a stress property called capillary potential, matric potential, or soil suction and they defined it as the work required to pull a unit mass of water away from a unit mass of soil exclusive of osmotic and other influences. Several factors affecting the relationship between soil suction and water content were discussed in detail by Spangler and Handy. These included temperature, dissolved salts, grain size, state of packing, angle of contact, and mineralogy of fines.

A soil-water characteristic curve is a form of expressing the relationships between water content and pressure potential. In an unsaturated soil column, when it is at equilibrium with free water, the water content decreases with height above the free water. Physically, the curve tells (at any given water content) how much energy (per unit quantity of water moved) is required to move a small quantity of water

from the soil. It indicates how tightly water is held in the soil. The area of soil physics lends itself to such topics, where references are available from such researchers as Hillel, Taylor, Ashcroft, Kirkham, Powers, and Childs just to name a few.

Soil water characteristic curves are divided into two types, namely,

- a. Sorption curves representing the water rise case.
- b. Desorption curves representing the drying or the draining case.

The difference between them is known as hysteresis. Hillel (1980) gave the following causes which may contribute to hysteresis:

1. The geometric nonuniformity of the individual pores which are generally irregular in shape voids and are interconnected by small passages. (This results in the ink bottle effect.)

2. The contact-angle effect, i.e., the greater the contact angle the larger the radius of curvature in an advancing meniscus than in a receding one. (A given water content will tend therefore to exhibit greater suction in desorption than in sorption.)

3. Entrapped air, which further decreases the water content of newly wetted soil. (Failure to attain true equilibrium can accentuate the hysteresis effect.)

4. Swelling and shrinking, or aging phenomena, which result in differential changes of soil structure, depending on the wetting and drying history of the sample.

Janssen and Dempsey (1981) stated that generally the desorption (the drying curve) is sufficient for most civil engineering uses and it is the most critical.

The soil-water characteristics curve can be determined using one or more of the following methods depending on the soil encountered.

- a. tensiometer method
- b. direct suction method
- c. pressure plate method
- d. centrifuge method

## 2.2 Deformation Characteristics of Subgrades Under Repetitive Loading

Monismith et al. (1975) developed a constitutive relationship between plastic strain and number of stress application. The relationship was represented by a power function as follows:

$$\epsilon^P = A N^B$$

where

$\epsilon^P$  = permanent strain

N = number of stress applications

A & B = experimentally determined coefficients

This equation was verified up to 100,000 stress applications. The authors also indicated that the subgrades of well-designed pavements are subjected to comparatively small stresses from conventional traffic loads (9,000-lb. wheel load). At stiffnesses in the asphalt bound layer larger than 200,000 psi, the vertical compressive stresses in the subgrade were less than 5 psi. At these stress levels, measurements of permanent deformation are not as precise as at higher stresses. Dempsey (1976) indicated that strength and stiffness parameters for aggregate materials are effected by moisture. However, the relative effect is influenced by the material's gradation, percent of fines and the degree of saturation.

Hicks and Monismith (1971) showed that increased saturation lead to reduced resilient moduli values for granular materials.

Barksdale (1972) found that the plastic strain in granular specimens increased an average of 68 percent when they were soaked. Seed et al. (1962) showed that pavements containing a saturated granular base layer displayed higher magnitudes of deflection than those pavements that were "dry."

Dempsey (1976) noted that unsoaked and soaked CBR values (a static type test) for granular materials were normally not significantly different, indicating the significance of repeated, dynamic loading conditions.

Shackel (1973) concluded that for a particular molding saturation, the resilient axial strains decreased linearly with increasing suctions. He also concluded that the cumulative, nonrecoverable (residual) axial strains decreased rapidly as the suctions increased.

Monismith et al. (1975) reported on two approaches which are available to consider rutting (permanent deformation) as a result of repeated traffic loading. One of the approaches involves limiting the vertical compressive strain at the subgrade surface to some tolerable amount associated with a specific number of load repetitions (Shell Criteria). By controlling the characteristics of the material in the pavement section through materials design and proper construction procedures (unit weight or relative compaction requirements) and by ensuring that materials of adequate stiffness and sufficient thickness are used so that the strain level is not exceeded, permanent deformation equal or less than some prescribed amount is thus assured. The other approach involves an estimation of the actual amount of rutting which

might occur using materials characterization data developed from laboratory tests. Chou (1977) stated that the major advantage of the first approach is that it could be used as a workable tool for the pavement design, and several agencies have introduced procedures based upon it.

Only the Criteria developed by Shell Oil Company will be considered in this study. Such Criteria can be utilized to ensure that permanent deformation in the subgrade will not lead to excessive rutting at the pavement surface. These Criteria may be thought to be associated with ultimate rut depths of the order of 3/4 inch.

The Shell Criteria are based on California Bearing Ratio (CBR) procedures and empirical correlation with results from the AASHTO Road Tests. These tests included single axle loads ranged from 2,000 to 30,000 lbs, including the standard 18,000-lb. The pavement section consisted of a surface course (Bituminous concrete 1 to 6 inches), a base course (a well-graded crushed limestone 0 to 9 inches), and a subbase layer (a uniformly graded sand-gravel mixture 0 to 16 inches). The testing covered a wide range of combinations of the pavement components. These combinations are still applicable today, which makes use of the Shell Criteria a reasonable one. To use the Shell Criteria, the asphalt concrete stiffness should range between 100,000 and 200,000 psi, and the Poisson's ratios of the materials in the pavement section should be in the range of 0.35 to 0.40.

In this study, the simulation of the soil-water retention in the subgrade soils will be used to characterize the permanent deformation due to repetitive loading.

## CHAPTER III QUESTIONNAIRE SURVEY

### 3.1 Background

A twenty item questionnaire was sent to the Departments of Transportation of all fifty states, to the District of Columbia, and to three Canadian provinces, to determine if they made use of highwater clearance specifications and if so, the bases for their criteria. A copy of the questionnaire and the cover letter are included in Appendix A.

Questions were designed to provide information on the existence of similar highwater clearance guidelines at the other Departments of Transportation, the development of such guidelines, consideration of soil type, duration of highwater level, determination of designated highwater level, common materials used for subgrade, capillary phenomena, measures taken to reduce the capillary saturation problem, equipment used for evaluating pavement deformation, criteria used for evaluating pavement deformation, and any current research or previous studies in the area of highwater clearance.

A total of 52 out of 54 departments (or 96 percent) responded. Most of the surveys were completed by pavement designers or drainage engineers. Their practical experience and sound judgment was evident from the responses.

### 3.2 Analysis of Responses

Since many of the survey questions elicited descriptive responses, the survey is not amenable to any type of computer analysis or even to any useful tabular listing of results. The responses have instead been summarized in the following ten paragraphs.

1. Fifteen departments have guidelines similar to those of the Florida Department of Transportation. Ten of the fifteen do not consider soil type. This suggests that these departments apply their guidelines uniformly to attain the same goals as the FDOT. In areas where frost is a problem, clearances of 4 to 6 feet from the finished grade to designated highwater are required, especially where the subgrade soil is silt or clay. Of those departments which do not have guidelines similar to those of the FDOT, some considered each project individually, some keep the highwater level below the subbase, while still others take measures to break the capillary rise. In all cases reported, experience and engineering judgment were used to develop the clearance guidelines. No testing procedures are followed in considering soil type.

2. Only six departments consider the duration of the highwater level. These departments are in the north central states. The spring season, with snow melt, is considered the critical time. For short highwater durations, they believe pavement damage will not result. For longer durations, they take protective action such as closing lanes or limiting vehicle loads.

3. Thirty-four of the departments responding place great importance on the determination of the designated highwater level. Some of the procedures mentioned make use of

- Nearby rivers or lakes.
- Borings.
- Interviews with maintenance personnel, mail carriers, local residents, etc.
- Reviewing previous records.
- Near bridge crossings use water surface profile program.
- Routing the 100 year flow through the bridge.
- Using National Flood insurance maps.
- Corps of Engineers as a source of information and records.
- Computer programs from Corps of Engineers, Soil Conservation Service, and USGS.
- Soil conservation runoff methodology.

4. The subgrade materials used by the different departments ranged from gravel and sand to silt and clay. The material depended on what was locally available with economic feasibility as the deciding factor.

5. Capillary rise was acknowledged by thirty-eight of the departments as one of the factors leading to a reduction of subgrade strength. Departments which use only gravel and coarse sand as subgrade material did not consider the effect of capillary rise on subgrade strength, simply because these soils exhibit very little capillary rise.

6. Measures taken to reduce capillary effects were

- Use of free draining materials with controlled percent of fines (maximum 10 percent passing #200).
- Use of lime treated subbase.
- Use of under-drains and adequate surface drainage.
- Increased subbase depth where suspect material was encountered.

7. Methods used for evaluating pavement deformation were

- Dynaflect.
- Road rater and profilometer.

- Benkleman Beam.
- Field measurement of rutting, and experimental sections.

The most commonly employed methods were the Dynaflect and the Benkleman Beam.

8. Thirty-one of the departments responding do not have specific criteria for evaluating pavement deformation. Twenty-one departments use the Asphalt Institute or the Shell Oil Company criteria. American Association of State Highway Transportation Officials (AASHTO) road data and Chevron research are also used.

9. No departments were involved in, or were aware of, any ongoing or previous studies on highwater clearance effects.

10. Some departments sent chapters of their design manuals and expressed their interest in the study by requesting copies of the final report.

## CHAPTER IV MATERIALS DESCRIPTION

### 4.1 Introduction

The Bureau of Materials and Research, FDOT, at Gainesville, provided four soils for the research. These represent the most commonly used fill soils in the State of Florida.

Ten bags of each material, weighing a total of approximately 500 pounds, were received. Visual identification was made in accordance with ASTM D 2488-84. A summary of this identification, the as-received water content, and the source of the materials by districts, are listed in Table 4.1. Figure 4.1 shows a map of the State of Florida and the location of the six FDOT districts.

Following the inspection and identification, the materials were air-dried by spreading on the laboratory floor and passing air over them with a fan. Each was then placed in a 30-gallon rubber container with two plastic liners for sealing. This maintained the material in the air-dry condition throughout the research.

### 4.2 Classification of Materials

The four soils were classified according to the AASHTO and the Unified Systems, using grain-size analysis (AASHTO T-11, T-27-82) and liquid and plastic limits (AASHTO T-89, T-90-81). Specific gravities (AASHTO T-100-75) were also determined.

Table 4.1 Visual Description and Identification of Project Soils in Accordance with ASTM D-2488-84

Soil No.	Location (District #)	Soil Description	Grain-Size Shape	Color	As Received Moisture	Odor
1	1	Sand	Fine Subangular	Tan	Wet	7%
2	2	Silty-Sand	Fine	Dark Gray	Dry	0.5%
3	3	Clayey-Sand	Fine	Brownish-orange	Dry	1.0%
4	4	Clayey-Silt	Fine grained	Light gray	Wet	60%
						Organic



Figure 4.1 Map of the State of Florida and the Location of the Six FDOT Districts

The results from the classification and specific gravity tests are presented in Table 4.2. Based on AASHTO classification, the following identification will be used throughout the text:

Soil #1	A-3
Soil #2	A-2-4
Soil #3	A-2-6
Soil #4	A-5

Figure 4.2 shows the grain size distribution curves for all four soils.

### 4.3 Soil Density-Water Content Relationships

The soil density-water content relationships were determined by performing "Proctor" compaction tests. The Standard Proctor test (AASHTO-99-81) was performed on the four FOOT subgrade soils.

Stabilized subgrade materials, made by mixing three parts subgrade soil and one part lime rock by weight, were tested using the Modified Proctor test (AASHTO-180-74). A summary of the compaction test results is included in Table 4.3, along with calculated values of the void ratio and the percent saturation at optimum water content and maximum density. The following equations were used to calculate the void ratio ( $e$ ) and the degree of saturation ( $S$ ).

$$e = \frac{G_s \gamma_w}{\gamma_d} - 1 \quad \text{and} \quad S = \frac{G_s w}{e}$$

where

$e$  = void ratio

$G_s$  = specific gravity

$\gamma_w$  = water unit weight

$\gamma_d$  = soil dry unit weight

$w$  = moisture content (percentage)

$S$  = degree of saturation (percentage)

Table 4.2 Summary of Classification Results for All Project Soils

	Sieve Number	Diameter (mm)	Soil Number			
			1	2	3	4
			% Passing			
Sieve Analysis	4	4.76	99.67	99.90	100	100
	10	2.00	99.51	99.88	99.38	99.95
	40	0.42	94.80	99.70	77.95	98.95
	60	0.25	70.87	99.05	59.27	97.99
	100	0.149	16.31	63.24	42.31	96.31
	200	0.074	4.10	10.46	28.85 <sup>a</sup>	94.50 <sup>b</sup>
Specific Gravity			2.61	2.62	2.68	2.55
Atterberg Limits	Liquid Limit		NP	NP	31	52
	Plasticity Index				11	8
Classification	AASHTO		A-3	A-2-4	A-2-6	A-5
	Unified		SP	SP-SM	SC	MH <sup>c</sup>

Note:

- a Hydrometer test results showed 6.95% silt size and 21.9% clay size.
- b Hydrometer test results showed 73.5% silt size and 21.0% clay size.
- c Hydrochloric acid added to the soil resulted in a violent reaction indicating the presence of calcium carbonate rather than organics in the soil.

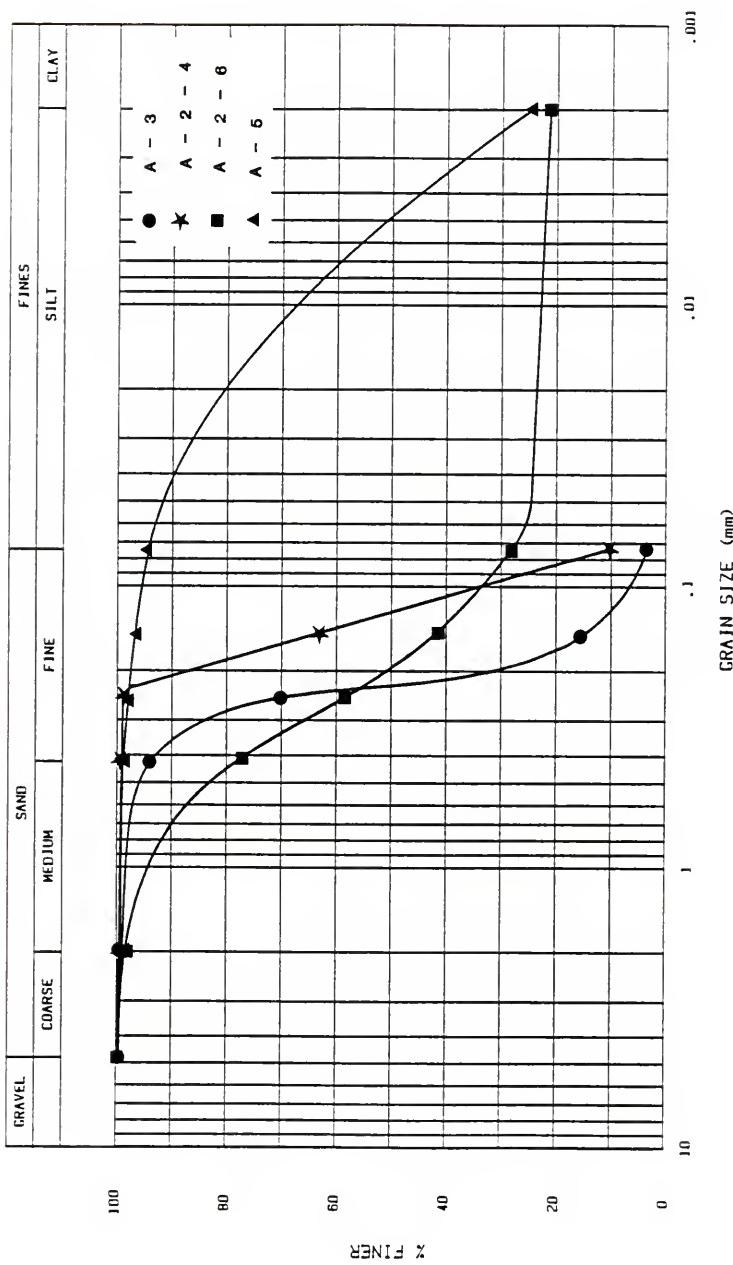


Figure 4.2 Grain Size Distribution Curves for All Project Soils

Table 4.3 Summary of Soil Density-Water Content Relationships

Soil No.	Soil Type	$\gamma_d$ [Maximum Dry Density, $\rho_{DF}$ ]		$w$ [% Water Content at Optimum] $T_{99}$ $T_{180}$		$e$ [Void Ratio at $\gamma_d$ Max] $T_{99}$ $T_{180}$		$S$ [% Saturation at Optimum] $T_{99}$ $T_{180}$	
		$T_{99a}$	$T_{180b}$						
1	A-3	102.2	110.8	15.61	12.50	0.59	0.47	69.1	69.4
2	A-2-4	104.9	112.2	13.27	11.57	0.56	0.46	62.1	65.9
3	A-2-6	120.5	127.9	11.80	9.50	0.39	0.31	81.1	82.1
4	A-5	73.7	88.7	36.41	25.50	1.17	0.79	79.4	72.6

## Note:

- a Method C of AASHTO T-99 was used for subgrades (5.5-lb hammer, 12-in drop, 25 blows, 3 layers, 1.7-in/layer, 4-in mold).
- b Method C of AASHTO T-180 was used for stabilized subgrades (10-lb hammer, 18-in drop 25 blows, 5 layers, 1-in/layer, 4-in mold).

Figures 4.3 through 4.6 show the test plots for the individual soils 1 through 4, respectively, for the Standard (T-99) and modified (T-180) proctor tests.

#### 4.4 Soil Mineralogy

The purpose of this analysis was to establish the nature of the clay minerals present in the project soils. Particular emphasis was placed on the presence of montmorillonite in the A-5 material, which exists in South Florida and may cause problems if used as fill.

Representative samples of the four soils were analyzed at the FDOT Bureau of Research and Materials in Gainesville, using X-ray diffraction techniques. Table 4.4 summarizes the results.

#### 4.5 Permeability Test

Permeability tests were performed on all soils. Each specimen was compacted at optimum water content (T-99) in a 6-inch diameter compaction mold. The mold was then fitted in a permeameter and the soil saturated. The constant head procedure was used for the A-3 soil, while falling head tests were performed on the other three soils. Table 4.5 summarizes the permeability test results.

#### 4.6 Soil-Water Retention Characteristics

The soil-water retention characteristics for the project soils were determined, using commercially available Tempe pressure cells. Two different size cells were used, one with a 3 3/8-inch diameter, the other with a 2 1/4-inch diameter. For the larger diameter apparatus, specimens were hand compacted directly in the cell. For the smaller

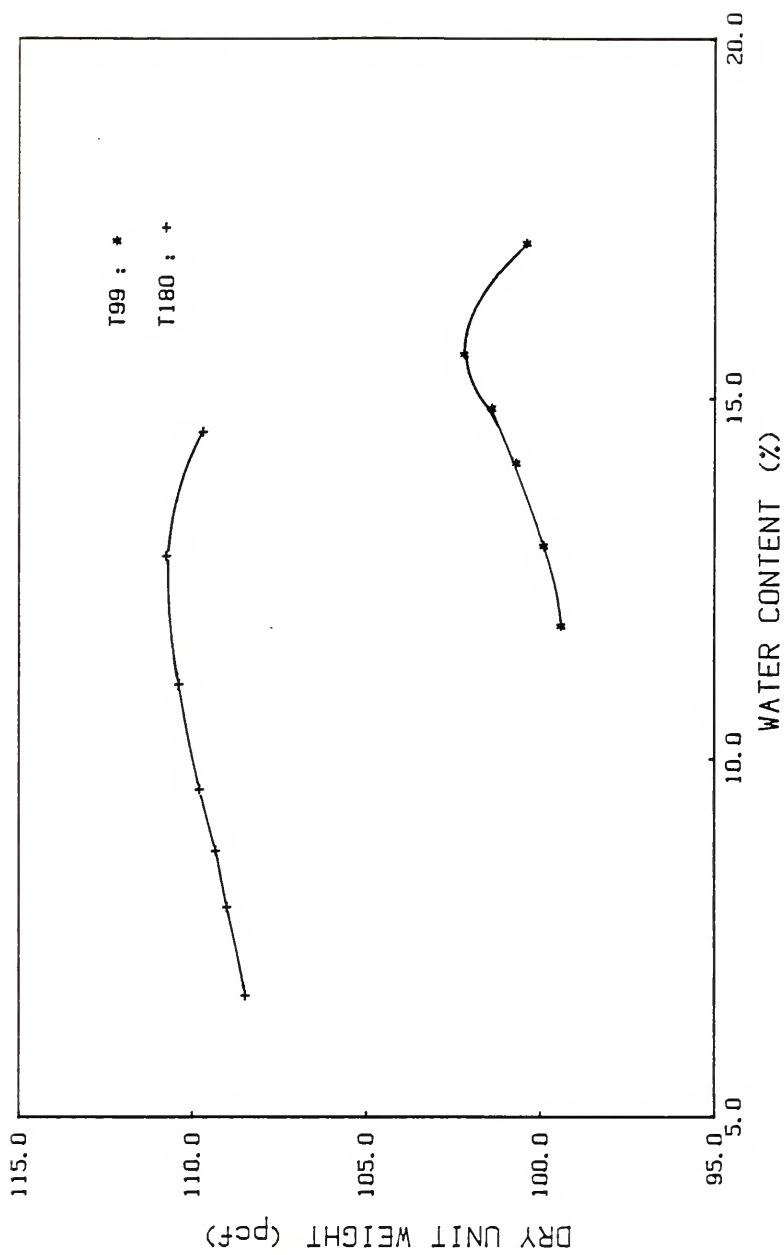


Figure 4.3 Soil density-Water Content Relationships for A-3 Soil

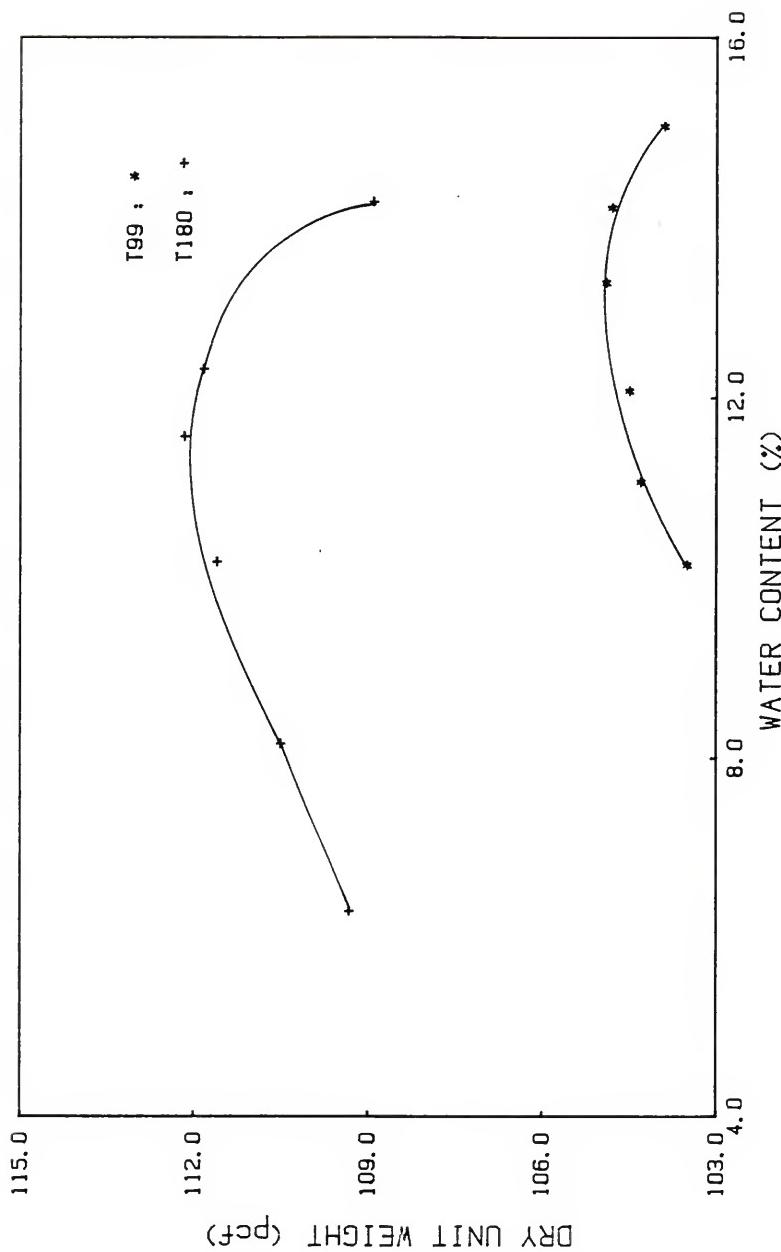


Figure 4.4 Soil Density-Water Content Relationships for A-2-4 Soil

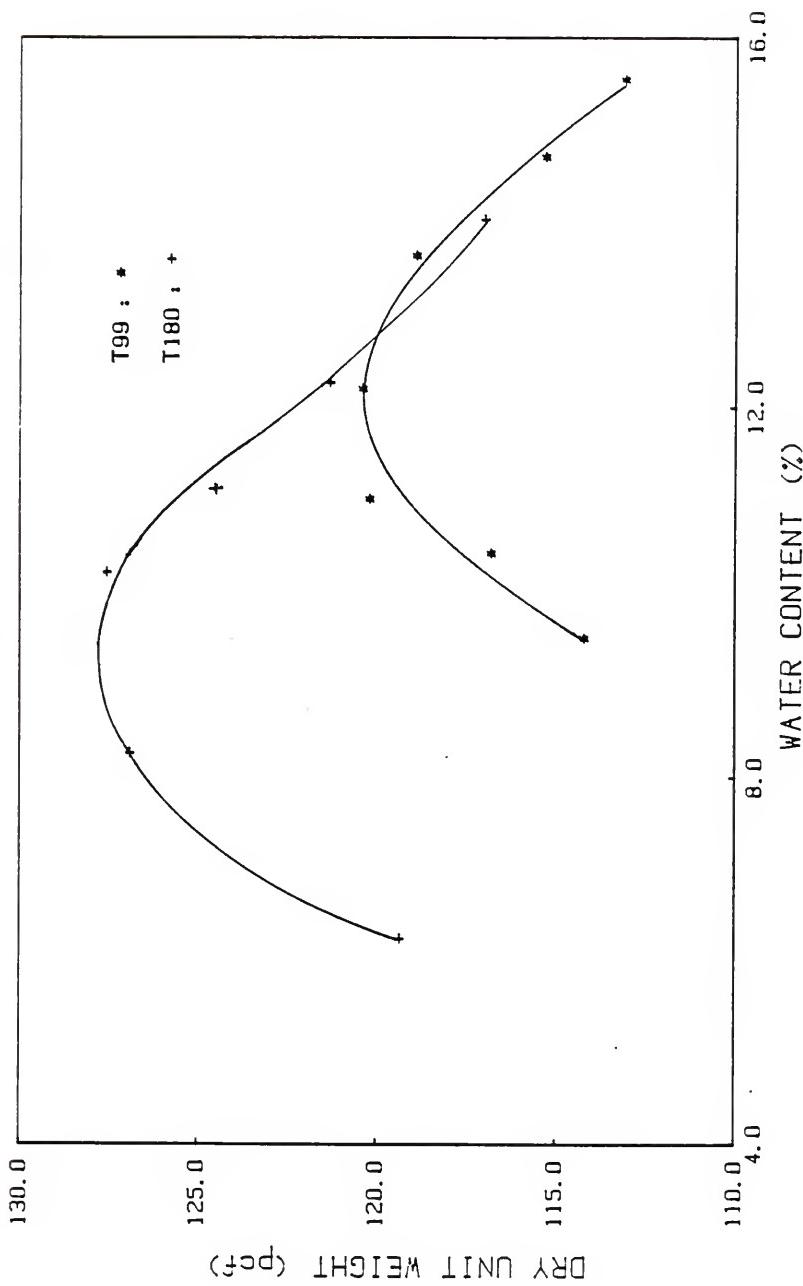


Figure 4.5 Soil Density-Water Content Relationships for Ar<sub>2</sub>-6 Soil

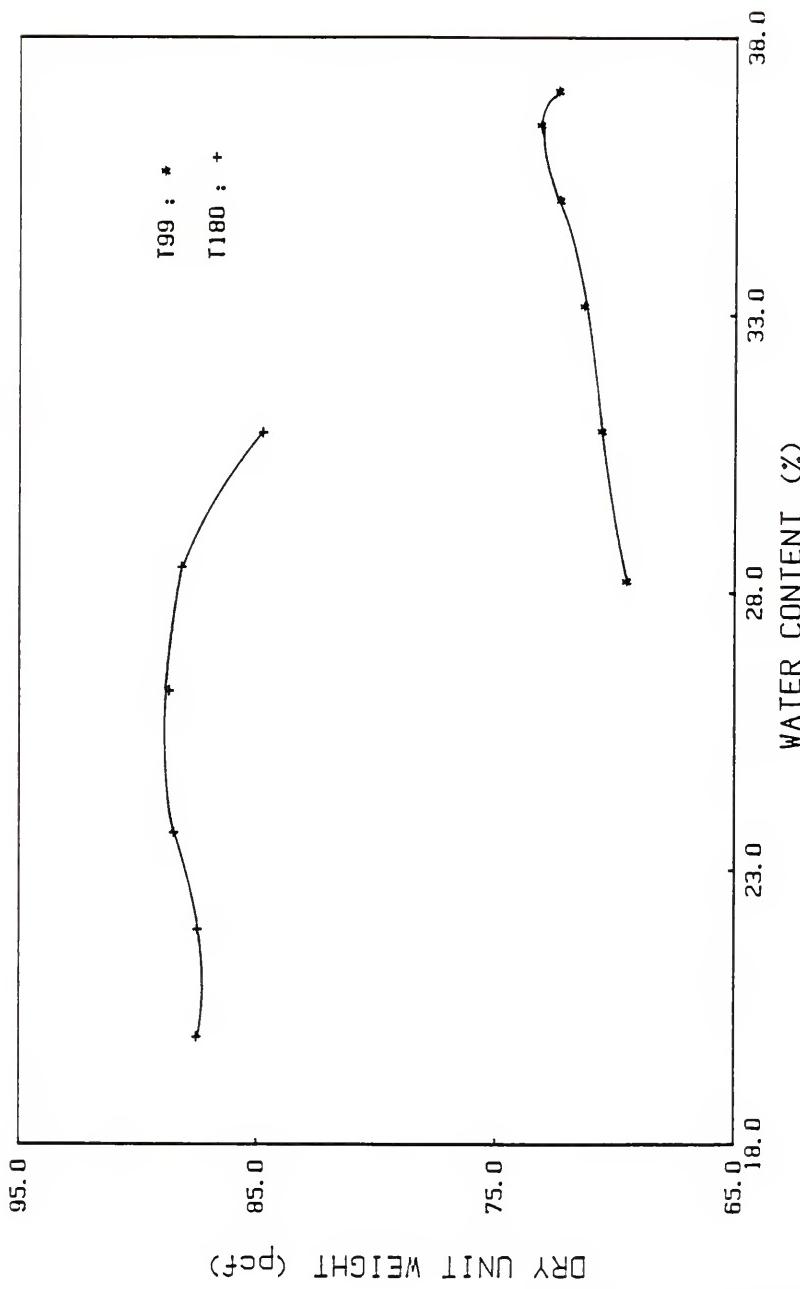


Figure 4.6 Soil Density-Water Content Relationships, for A-5 Soil

Table 4.4 Mineralogy Analysis of Clay Fraction

Soil No.	Soil Type	Clay Minerals Present
1	A-3	----
2	A-2-4	Kaolinite, Chlorite
3	A-2-6	Chlorite
4	A-5	Kaolinite, Montmorillonite

Table 4.5 Summary of Permeability Test Results

Soil No.	Soil Type	Type of Test	Coeff. of Permeability
			cm/sec
1	A-3	Constant head	$7.3 \times 10^{-4}$
2	A-2-4	Falling head	$1.1 \times 10^{-5}$
3	A-2-6	Falling head	$4.8 \times 10^{-7}$
4	A-5	Falling head	$6.4 \times 10^{-6}$

cell, specimens were obtained using a thin wall tube and sampling from a soil mechanically compacted in a standard 5-inch mold. Densities obtained were within 2 percent of the T-99 maximum dry density.

The testing was performed at the Soil Physics laboratory of the Soil Science Department, University of Florida. After preparing the soil specimen as described above, air was purged from under the plate by injecting water through the bottom nipple. The lid was then set on and the cell placed in a pan of water to saturate. The cell inlet tube was then connected to an air pressure source and the cell allowed to drain, initially under zero pressure. The cell was weighed daily. When the change in weight was less than 0.02 g for two consecutive days, equilibrium was assumed to exist, and a pressure of 3.5 cm of water was applied. This process was then repeated for pressures of 20, 30, 45, 60, 80, 100, 150, 200 and 345 cm of water, and at each stage the weight of the cell and sample was determined. The air pressure was measured by means of a water column. After reaching equilibrium under the 345 cm pressure, the apparatus was disassembled and the dry weight of the specimen determined after drying in an oven for 24 hours at 105° C. Knowing this weight, the water contents at the different pressures applied during the test can be computed. The system and procedure used are similar to those reported by Janssen and Dempsey, 1980. The photograph in Figure 4.7 shows the two sizes of Tempe cells. Figure 4.8 shows the setup during testing, including the water columns to the left. The Tempe cell utilizes pressure rather than suction to drive the water out of the soil pores and reach static equilibrium at the specific air pressure applied.

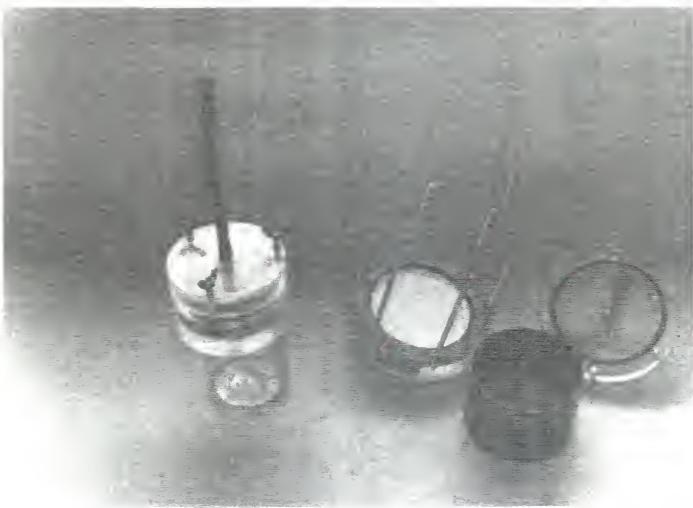


Figure 4.7 Different Size Teflon Cells

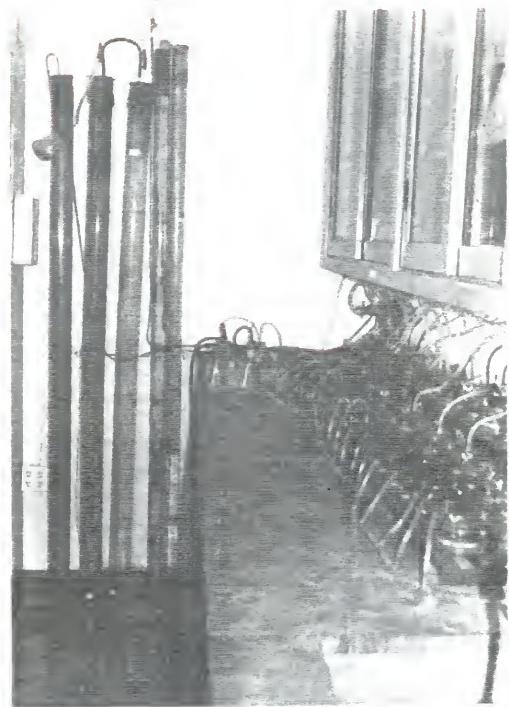


Figure 4.8 The Tempe Cell Setup

Appendix B contains a summary of the data collected from the soil-water retention tests on both hand compacted and mechanically compacted specimens. Hand compacted specimens were tested because their fabric and density would match those obtained in the column study, which will be described later. The tests using mechanically compacted specimens will be the references for the subsequent repetitive loading testing at varied water conditions. Figures 4.9 through 4.12 show plots of gravimetric water content ( $w$ ) versus distance above the water table for the four project soils compacted both mechanically and using the hand tools.

It is also common practice to plot the above relationships using the volumetric water content  $\theta$ . This is defined as the fraction of the soil volume occupied by water (Yong and Warkentin 1975).

$$\theta = \frac{V_w}{V_s + V_v} \text{ or } \frac{V_w}{V_t}$$

where

$\theta$  = volumetric water content

$V_w$  = volume of water

$V_s$  = volume of solids

$V_v$  = volume of voids

$V_t$  = total volume

The relationship between  $w$  and  $\theta$  is

$$\theta = \frac{w}{100} \times \gamma_d \times \frac{1}{\gamma_w}$$

where

$w$  = gravimetric water content

$\gamma_d$  = dry density =  $\frac{W_s}{V_t}$

$\gamma_w$  = unit weight of water

$W_s$  = weight of solids

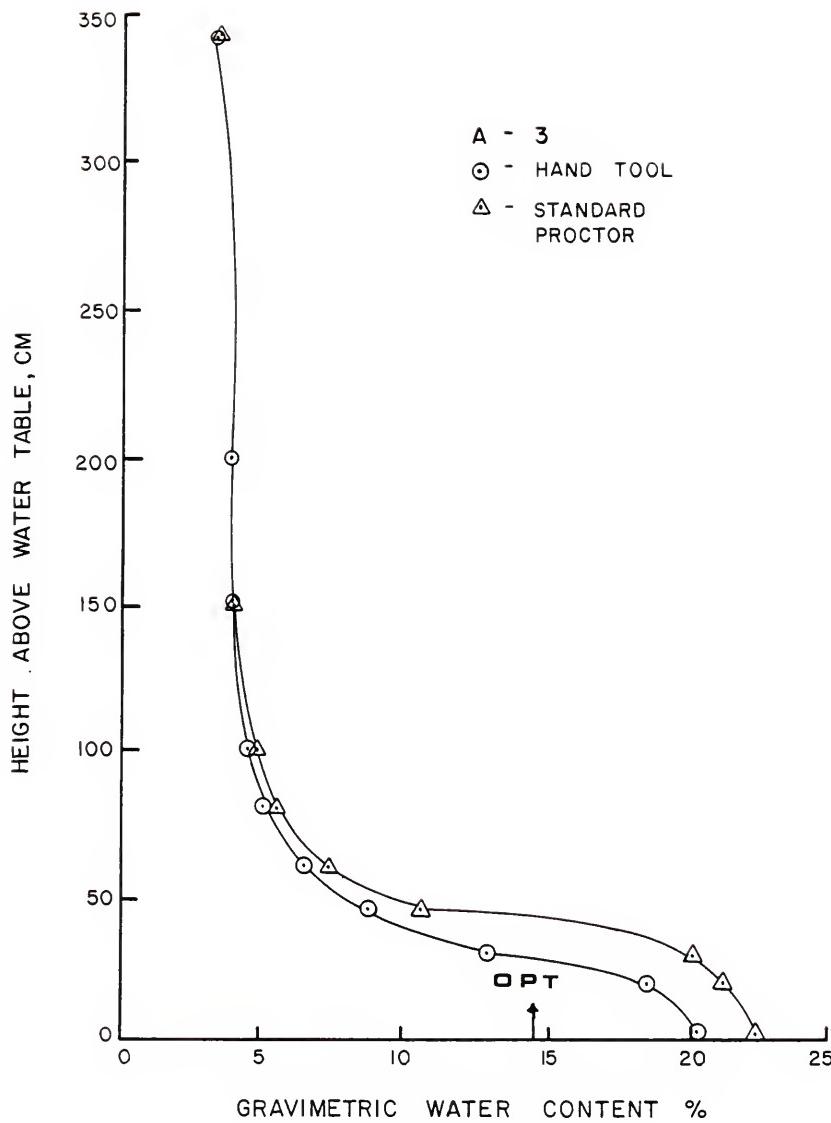


Figure 4.9 Gravimetric Water Content Versus Height Above Water Table for A-3 Soil

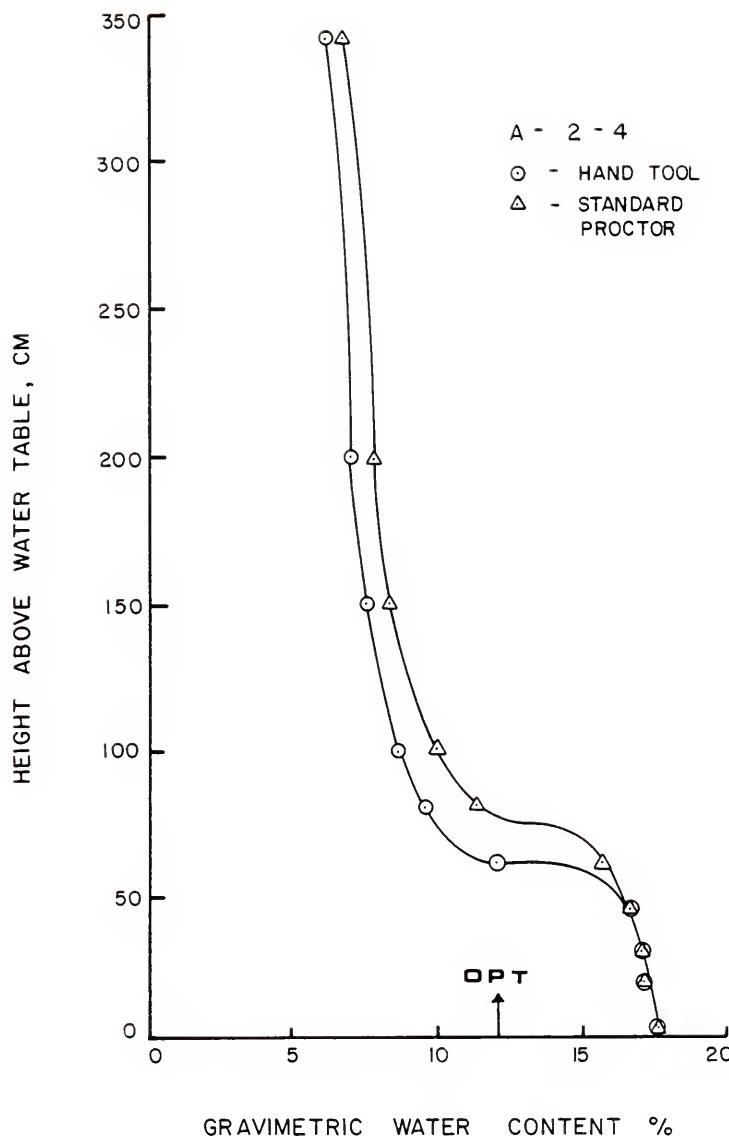


Figure 4.10 Gravimetric Water Content Versus Height Above Water Table for A-2-4 Soil

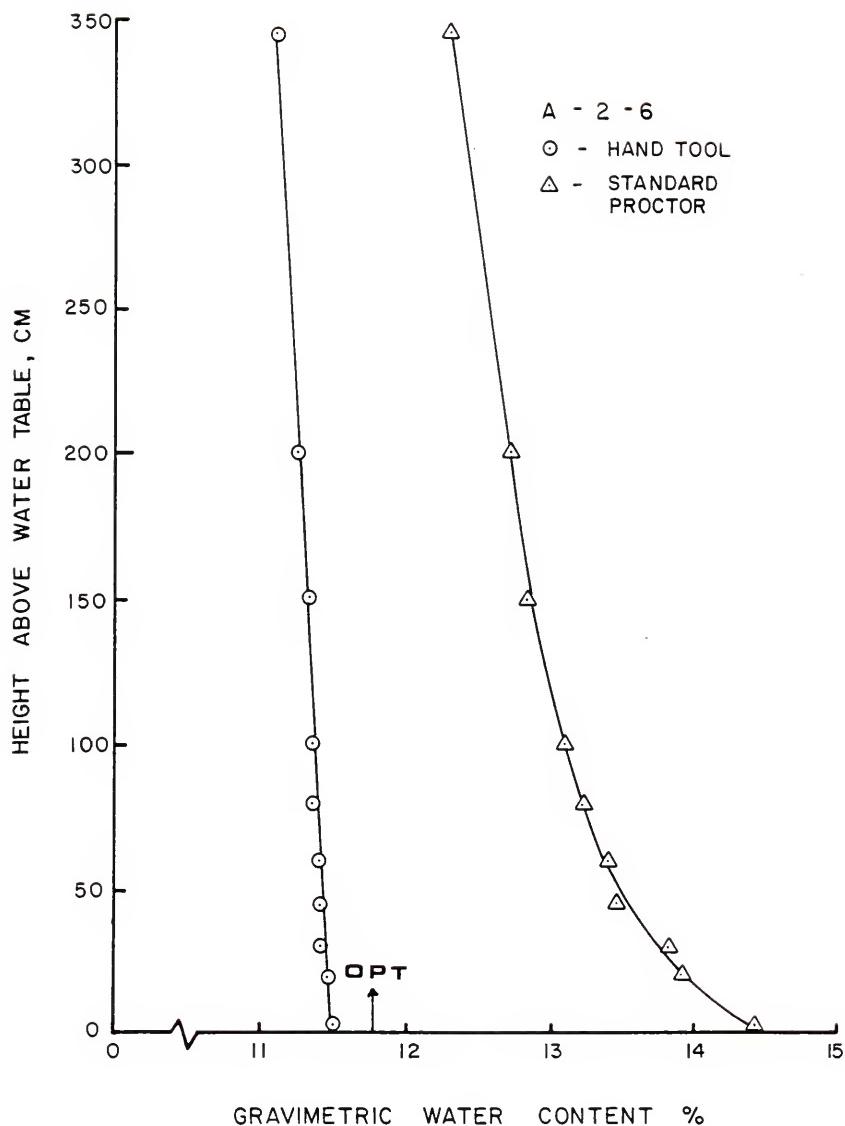


Figure 4.11 Gravimetric Water Content Versus Height Above Water Table for A-2-6 Soil

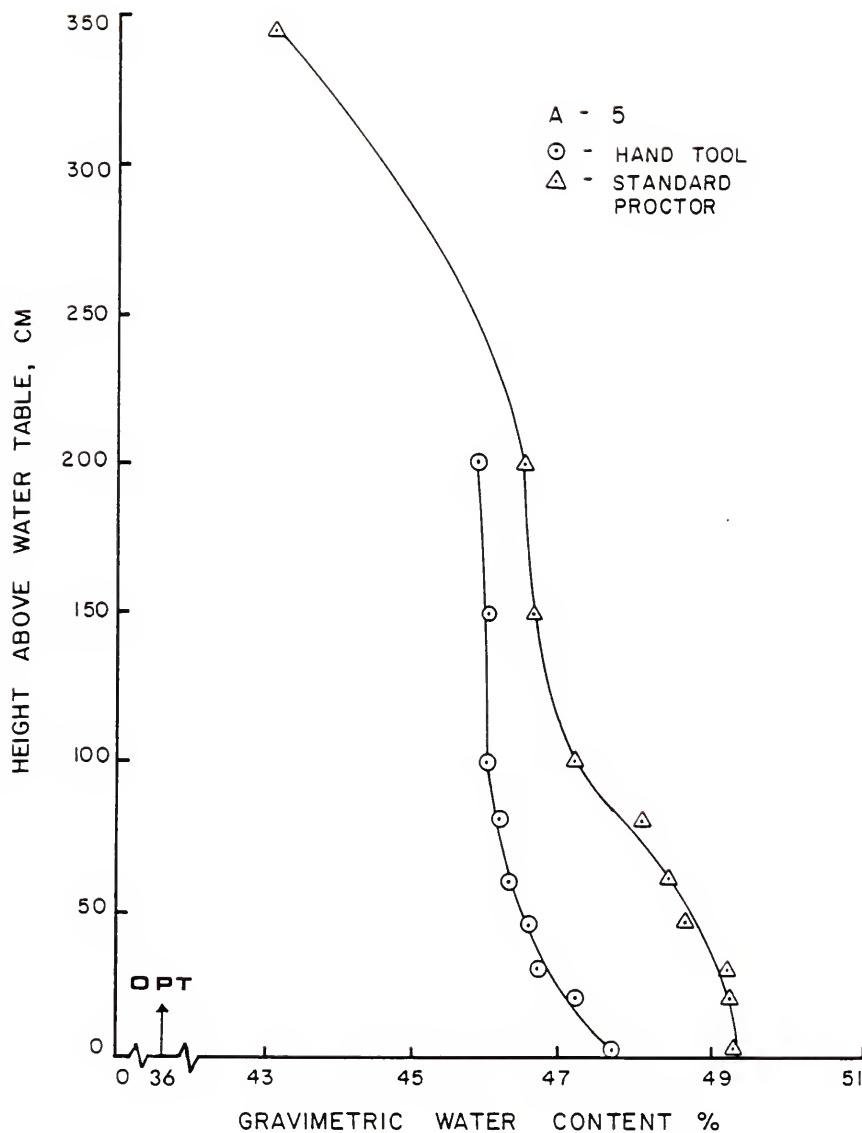


Figure 4.12 Gravimetric Water Content Versus Height Above Water Table for A-5 Soil

In the testing only a limited range of soil suction (to 1/3 bar, about 11.32 feet of water) was necessary to represent the soil-water at static equilibrium with a shallow water table. Because soil suction is given in units of water head, it is equal to the distance above the water table at equilibrium condition.

The A-3 soil-water retention curve in Figure 4.9, shows a distinct decrease in water content at a small height above the water table. Between 30 and 45 cm, the specimen water content decreased from 20.27 to 10.63 percent. On the other hand, an almost constant water content was reached at a height between 150 and 345 cm where the variation in water content was only from 4.20 to 3.56 percent. This characteristic shape is due to the greater percentage of the larger pore sizes in sand, which would empty at low suction values. The hand compacted specimen and the mechanically compacted specimen showed similar trends, but with a small difference at low height above the water table. The difference could be due to variation in the structure of the soil, resulting from the method of compaction and differences in density. The term structure is used to cover the size, shape, and arrangement of the particles and voids.

The A-2-4 soil-water retention curves, in Figure 4.10, show a trend similar to that found in the A-3 soil, except that they have distinct capillary saturations of 60 and 45 cm for the mechanically and hand compacted specimens, respectively. The overall fine, uniform gradation, and the presence of 11 percent fines ( $< 0.074$  mm) in the A-2-4 soil could be responsible for the development of the capillary saturation. Again, the differences between the two curves are probably due to variations in structure. Between 150 and 345 cm there was, again, no significant change in water content.

The A-2-6 material, which is a well-graded soil, shows in Figure 4.11 a gradual and uniform decrease in water content with increasing height above the water table. The difference between the mechanically and hand compacted specimens was very pronounced, although the difference in densities was the same as in the A-3 and A-2-4 soils. The reason for the difference is the large number of layers in the hand compaction (twelve) versus (three) in the mechanical compaction, resulting in differences in the developed structure. The mechanically compacted specimen did not reach the optimum water content (11.8 percent) within the testing range of 345 cm (11.32 ft). It would require much higher suction to reach this value. This observation is in agreement with what has been reported by Kersten (1944), and Janssen and Dempsey (1980). They found in field studies that a significant number of subgrade soils, which had clay contents, had water contents which were above the optimum water content. The number of such cases increased directly with increasing clay content. The A-2-6 soil tested contained 21.9 percent clay.

The characteristic curve for the A-5 soil, which is predominantly silt, is shown in Figure 4.12. The water contents remained near saturation up to a height of 200 cm. Then a relatively rapid reduction in water content took place. This specimen also did not reach optimum water content (36.4 percent) within the range of the test (1/3 bar, 11.32 ft). The A-5 soil also has 21 percent clay content. Due to an experimental problem with the last reading at 345 cm for the A-5 hand compacted specimen, the water content was not considered in the analysis.

For both the A-2-6 and A-5 soils it seems that the method of compaction and the number layers, used to obtain the desired density, had great effect on the shape of the characteristic curve and the amount of water retained at any height above the water table.

Figure 4.13 shows a cumulative graphical presentation of the soil-water characteristic curves for all project soils mechanically compacted to the Standard Proctor. The distinction in shape for each soil-water characteristic curve is evident in Figure 4.13. The A-3 curve is located to the far left followed by the A-2-4, then the A-2-6, and finally to the far right, the A-5 curve. It was noted that the amount of fines ( $< 0.074$  mm) increased from left to right. These curves can be helpful in recognizing the soils that are susceptible to large changes in water content as a result of changing the position of the water table.

The mechanical behavior of subgrade soils is affected by the presence of water and the development of capillary potential above the water table. It is therefore essential in any study to simulate the field condition for the subgrade soils as it exists, in-service, under the pavement, and in equilibrium with the designated water table. The soil-water characteristic curves provide the necessary parameters to achieve this simulation.

#### 4.7 Static Triaxial Compression Tests

Standard static triaxial compression tests were performed on all project soils in order to define failure envelopes. Tests were performed consolidated drained and under strain controlled conditions. Soil specimens, which were 4 inches in diameter and 8 inches high, were

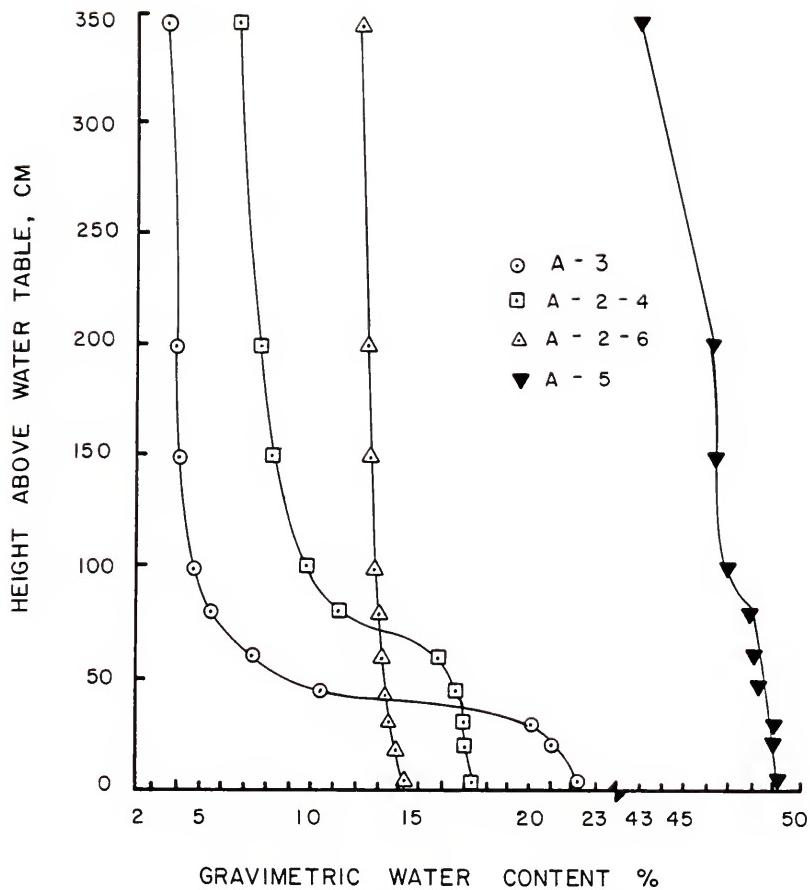


Figure 4.13 Retention Characteristic Curves for all Project Soils Compacted to Standard Proctor

prepared by mechanical compaction to the Standard Proctor at optimum water content.

The triaxial tests provide the angle of internal friction,  $\phi$ , and the cohesion,  $c$ . Table 4.6 summarizes the testing results.

Table 4.6 Triaxial Static Compression Test Results

Soil Type	Confining Pressure $\sigma_3$ psi	Deviator Stress (peak) $\sigma_1 - \sigma_3$ psi	Cohesion $c$ psi	Angle of Internal Friction $\phi$ degrees
A-3	2	11.50		
	10	42.50	1.5	40.0
	15	58.50		
A-2-4	2	21.00		
	5	32.50		
	10	44.70	4	36.0
	15	58.50		
A-2-6	2	26.40		
	10	46.80	6	34.0
	15	60.90		
A-5	2	40.90		
	5	49.50		
	10	61.00	10	33.0
	15	72.20		

## CHAPTER V EQUIPMENT AND PROCEDURES

### 5.1 Column Study

To study capillary effects in the subgrade soil, full height soil columns were prepared. Each column consisted of sections of lucite cylinder, 5 5/8 inches I.D. and 6 inches high, stacked on top of one other to provide the desired height. Holes 7/8-inch in diameter were drilled in each section for tensiometers which measure soil suction, Figure 5.1. Each column rested on a base to which a tall cylinder could be connected to provide upward flow saturation of the soil. A simple inverted flask was used to keep the water table at a fixed level, as shown in Figure 5.2. Figure 5.3 is a photograph showing the setup during testing. Mercury manometers, Figure 5.4, connected to the tensiometers measured the developed suctions.

The test setup was intended to represent the subgrade and stabilized subgrade layers in a pavement system. The subgrade soil was compacted to AASHTO T-99 and the stabilized subgrade to AASHTO T-180. This was achieved by hand compacting a determined weight of soil into a known volume using the tools shown in Figure 5.5. The tensiometers were inserted during compaction to ensure intimate contact between the porous cup and the surrounding material. The primed tensiometers were then connected to the mercury manometers. Each column was saturated from the bottom up by connecting the bottom of the column to a tall cylinder. The water level in the tall cylinder was maintained at the same level as



Figure 5.1 Lucite Column Section and Tensiometer

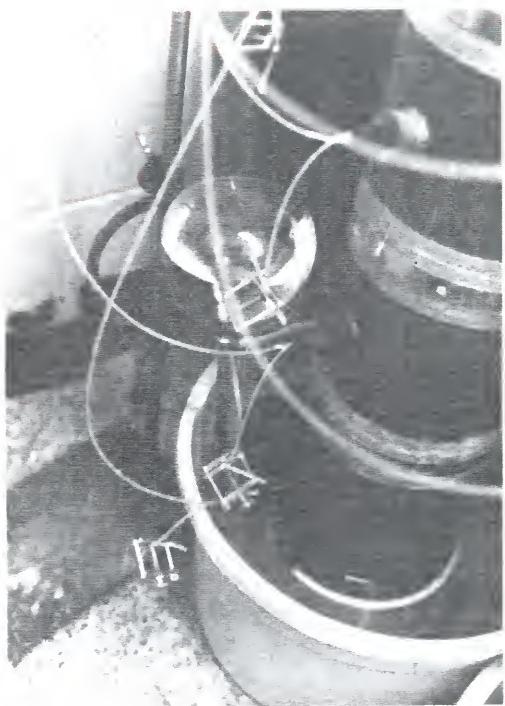


Figure 5.2 Inverted Flask to Maintain Fixed Water Level

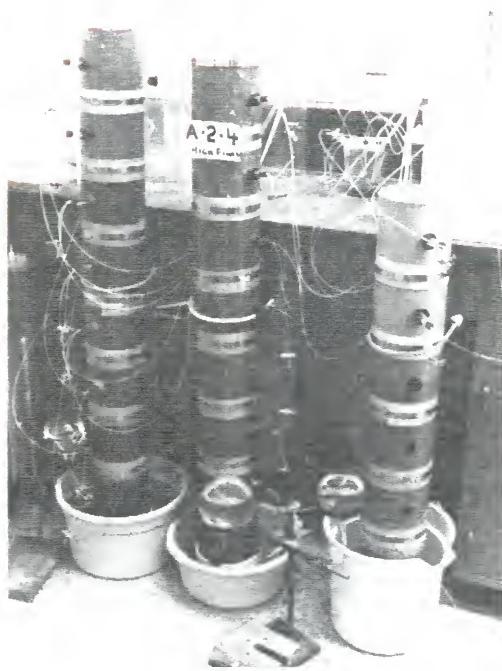


Figure 5.3 Column Setup During Testing

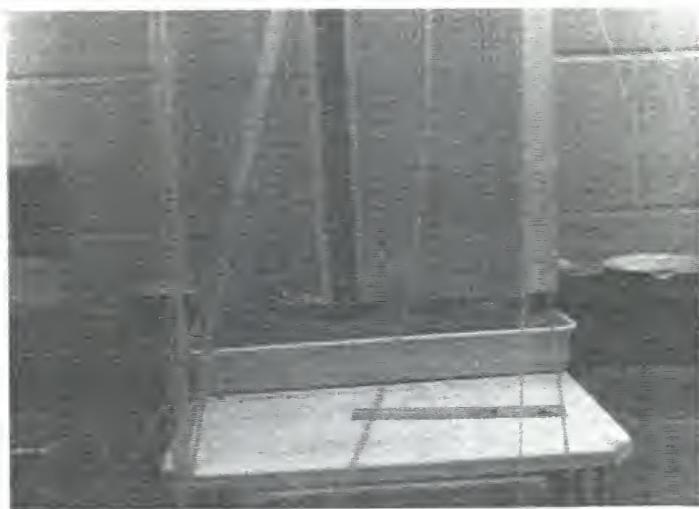


Figure 5.4 Mercury Manometers for Measuring Soil Suction in the Column

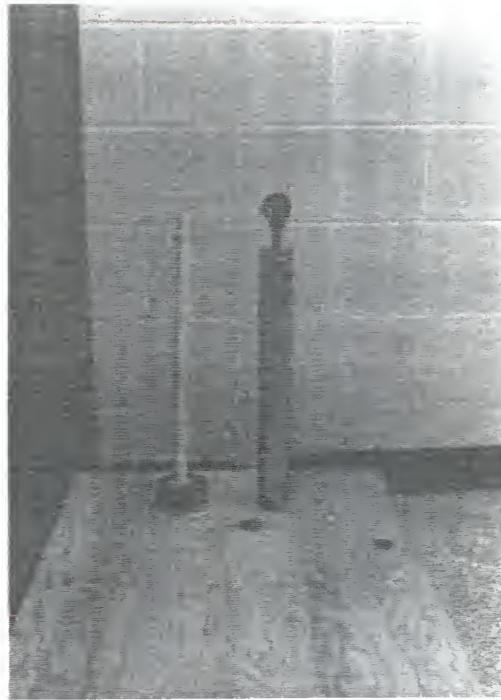


Figure 5.5 Hand Compaction Tools

the top of the soil in the column. The upward infiltration of water into the soil simulates a rising water table in a pavement system due to a heavy rain storm. This represents the worst condition expected in the field. When the tensiometers in the column indicated zero tension, the column was considered to be saturated. Free drainage was then allowed to occur until the soil reached a condition of static equilibrium with the water table at the bottom of the column. The drained water was allowed to overflow, while the water table was maintained at a fixed level using a simple inverted flask. The water distribution in the soil column at equilibrium is near saturation at the water table level and decreases with height above the water table. When the soil is at some water content less than saturation, it draws water through the porous walls of the tensiometer cup. As a result, a negative pressure is produced in the cup water and the mercury rises in the closed leg of the manometer. This process continues until the soil and the cup water have the same pressure deficiency or tension. The pressure deficiency, which is numerically equal to the capillary potential, may be determined directly by reading the height of mercury rise, above the mercury sump, in the manometer leg.

In this study only the retention case, which is the more critical condition, was considered. In it the soil at a certain height above the water table would have a higher water content than in the wetting up case. The higher the water content the less stable the subgrade. The relationships developed between suction and water content will be compared later to the results obtained using the Tempe cells. The column study is described in detail in a thesis by Gallet, 1986, as part of the same project.

## 5.2 Repetitive Load Testing

### Specimen Preparation and Conditioning

The air dried material was sieved through a #4 sieve (4.76 mm) and mixed with enough water to provide the optimum water content previously determined in accordance with AASHTO T-99. The mix was then compacted (5 equal layers, 26 blows/layer, 5.5-lb hammer, 12-inch drop) to give a specimen size of 4-inch diameter and 8-inch height, Figure 5.6. A lucite cylindrical mold was used so that the specimen could be observed throughout the testing. These specimens, at optimum water content, represent the subgrade as built. Repetitive loading triaxial tests were performed on such specimens of all four subgrade materials to obtain the as built deformation characteristics.

The water content at any location in the subgrade soil will not remain at the optimum compacted value, but will change until it comes into equilibrium with groundwater conditions. In this research only the more critical case of a draining soil after saturation was considered. Section 5.1 described this for the column testing. To prepare samples for repetitive loading triaxial testing, specimens were prepared as described above for the optimum condition, then conditioned to bring them to the desired water content and soil suction state.

The lucite mold containing the compacted specimen was first fitted with a perforated base, and filter paper at the bottom and a collar at the top. Threaded rods held the collar and base together. The mold assembly was then placed in a deep sink in which water was allowed to rise to the top of the mold and was maintained at that level. Water could enter the specimen only from the bottom. The mold top was covered with filter paper and a perforated light weight plate. When water drops

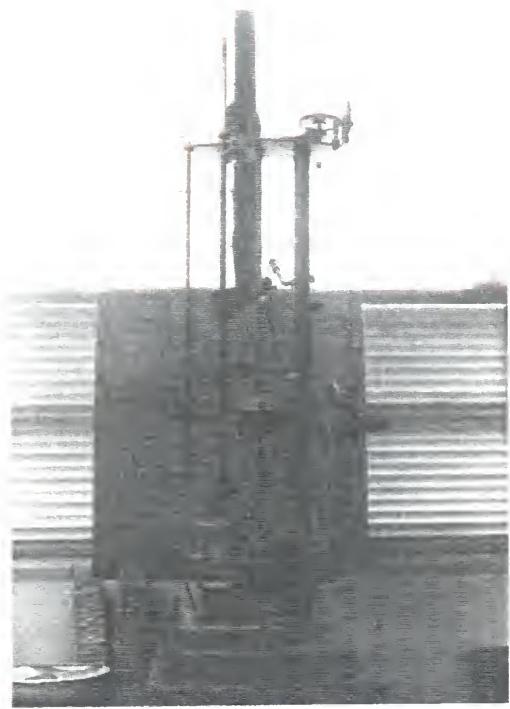


Figure 5.6 Lucite Mold and Compaction Equipment

were observed on the perforated top, the cover and filter paper were removed and the specimen visually inspected. If a glaze of water was observed on top of the specimen, it was considered to be saturated. The A-3 (sand) took only a few hours to saturate. On the other hand, the A-2-6 (Clayey sand) took almost 8 days. Figure 5.7 shows a mold assembly in the sink during saturation. Following saturation, the water in the sink was allowed to drain slowly, simulating water drawdown in the field. The mold assembly was then lifted out of the sink and the excess water allowed to drip for at least 30 minutes. Figure 5.8 shows a mold assembly during this phase. When no more dripping was observed, the assembly was taken apart and the mold, filled with the saturated soil, was weighed. Knowing the weight and volume of the empty mold, the unit weight  $\gamma$  of the specimen could be determined. Since the dry unit weight  $\gamma_d$  was also known, from the initial compaction, the water content could be determined based on

$$\gamma_d = \frac{\gamma}{1 + w}$$

Knowing the water content ( $w$ ), the void ratio ( $e$ ), and the specific gravity ( $G_s$ ) of the soil, the degree of saturation ( $S$ ) could then be calculated

$$G_s w = S e$$

This was the assurance that the specimen was saturated. When satisfactory saturation was achieved ( $S$  greater than 98 percent) the mold was fitted with a pressure plate (rated at 1 bar) a lucite cup with two drains, a small reservoir and a water manometer. O-rings and silicon grease were used to seal any gaps between the mold and the lucite cup. The upper end of the mold was covered with a perforated lucite plate to allow air to enter the specimen. Connecting rods were used to hold the

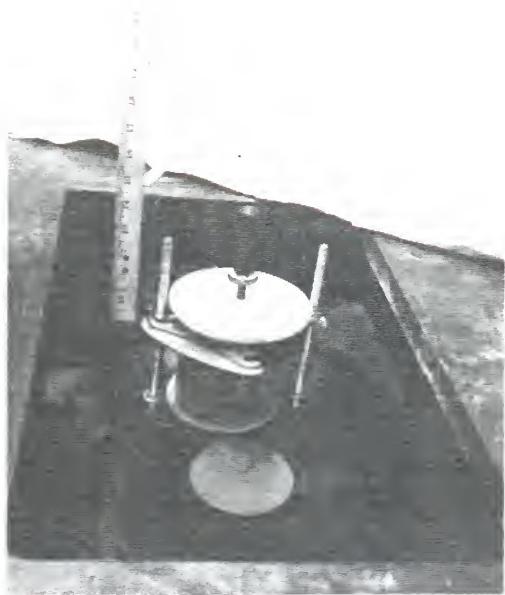


Figure 5.7 Mold Assembly in the Deep Sink During Saturation

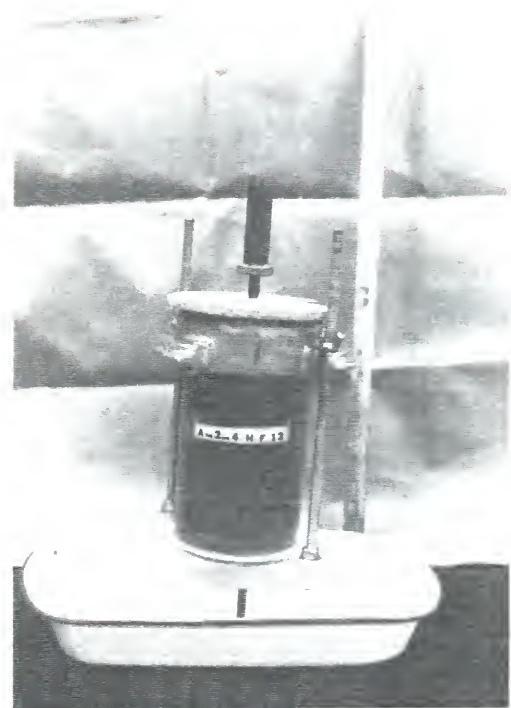


Figure 5.8 Mold Assembly During Free Drainage

cup, mold, and the cover together as one unit. The arrangement described above is essentially a large Tempe cell, except it uses a negative column rather than air pressure to condition the specimen. Figure 5.9 shows the components of the conditioning cup. The idea behind the conditioning cup is that the saturated specimen is subjected to a negative pressure through the pressure plate and the hanging column. When equilibrium is reached, the water content of the specimen is representative of the water content of a similar specimen in the subgrade at a height above the water table, equal to the length of the negative column. Figure 5.10 shows a specimen during conditioning.

When the specimen, under the effect of the applied negative column, ceased to drain water for 48 hours, it was considered to be at equilibrium.

The mold assembly was then taken apart and the wet unit weight and water content determined as before. At this point, a decision was made on whether the specimen had actually reached the target water content. The target water content was obtained from the soil-water characteristic curve, that is, the corresponding water content at a certain height above the water table. If the water content was not satisfactory, the specimen was refitted with the conditioning cup and the process repeated until the target water content was obtained. At that point the specimen was ready for the repetitive load testing.

Due to the size of the specimen, 4-inch diameter and 8-inch height, and the limited distance above the water table to be investigated as a fill height, attaining the target water content throughout the entire specimen was impractical. For this reason, it was decided that if the middle 4 inches of the specimen was within 1 percent of the target value,



Figure 5.9 Components of the Conditioning Cup

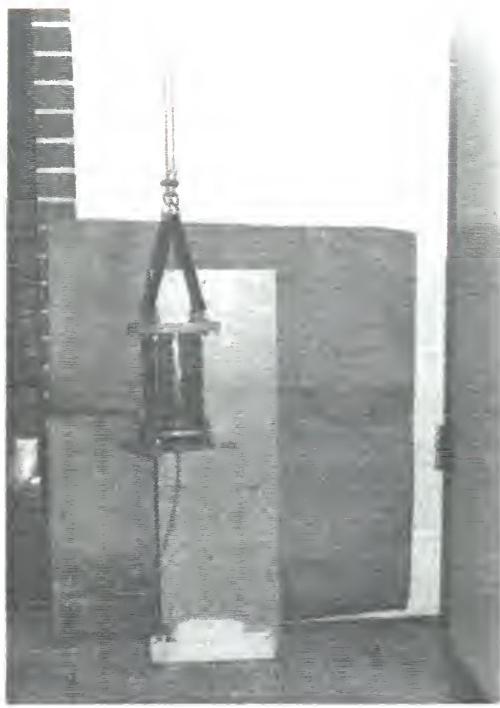


Figure 5.10 Specimen During Conditioning

the specimen was considered acceptable. Achieving this criterion required considerable time (up to 45 days in some cases).

#### Repetitive Loading Equipment

The repetitive loading equipment consisted of a triaxial chamber capable of accommodating a 4-inch by 8-inch specimen, vertical and lateral deformation measuring equipment and a vertical loading source. The external loading source was a closed loop electro-hydraulic system manufactured by MTS Systems Corporation. An electronic load cell measured the axial forces. The axial and radial deformation measurements were made using two pairs of linear variable differential transformers (LVDT's). These were mounted on a pair of expandable clamps which contacted the specimen through four wide feet on each clamp (see Figure 5.11). Air was used as the chamber fluid and was monitored with conventional pressure gauges. Signal excitation, conditioning, and recording equipment provided for simultaneous recording of the axial loads and deformations. The LVDT's were wired so that the average signal from each pair was recorded. Figure 5.12 shows an overall view of the equipment used. Figure 5.13 shows a close up to the MTS System, load cell and the triaxial chamber.

#### Testing Procedure

The objective of the repetitive load testing was to characterize the permanent deformation of the project soils at both optimum and varied water conditions. The testing procedures and equipment were the same in both cases. The specimen was extruded using a hydraulic jack, then transferred to the triaxial chamber base plate. Porous stones were used



Figure 5.11 Clamps With LVDT's for Measuring Lateral and Vertical Deformations

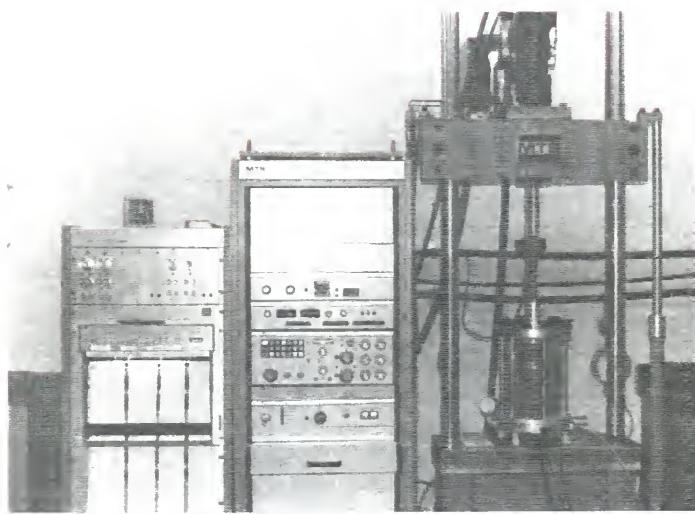


Figure 5.12 An Overall View of the Repetitive Loading Test Equipment

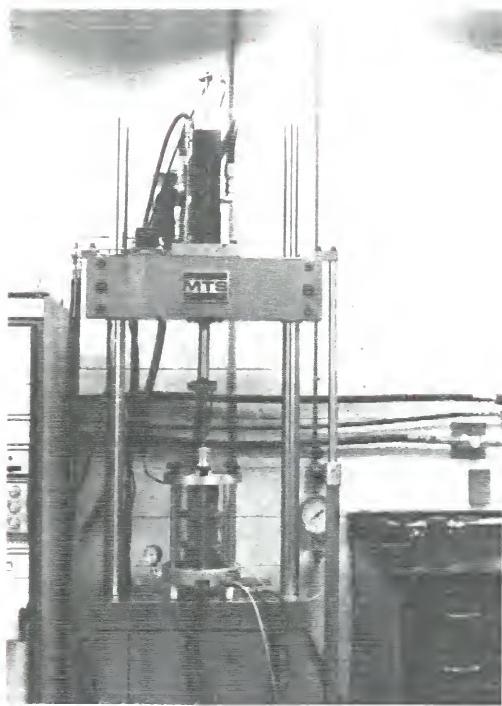


Figure 5.13 The MTS System, Load Cell and Triaxial Chamber

on top and at the bottom of the specimen. A rubber membrane was placed over the specimen, using a membrane stretcher, and secured to the top and bottom platens using O-rings. The membrane was marked with a felt-tip pen at the 2- and 6-inch heights on opposite sides ( $180^\circ$  apart) of the specimen. This was to aid in the placement of the clamps at the ends of the middle 4 inches of the specimen. The diameter of the specimen was measured, using a long-jaws caliper, at the 2-, 4- and 6-inch heights and the specimen tilt was checked using an air bubble level. The LVDT clamps were then placed at the 2- and 6-inch heights (see Figure 5.14). The vertical and horizontal LVDT's were zeroed and the chamber cylinder placed and connected to the chamber base plate using 6 tie rods. The loading piston was inserted through the top of the chamber cylinder and the load cell lowered until full contact with the loading piston was made through a steel ball. A confining pressure of 2 psi was applied for 30 minutes under drained condition.

The MTS controls were set to give a 0.1 sec load on, and 0.9 sec load off. The pulse wave was haversine. Knowing the specimen diameter and the stress desired, the load could be calculated and the controls set accordingly. The confining pressure was maintained at 2 psi in all tests. Dynamic conditioning of the specimen was performed to eliminate the end imperfection of the specimen, to allow for better seating of the porous stones, and to eliminate the effects of the interval between compaction and loading. This dynamic conditioning consisted of first applying 200 load repetitions of a 2 psi deviator stress ( $\sigma_1 = 4$  psi and  $\sigma_3 = 2$  psi). The axial load was then incremented by 2 psi after each 200 repetitions until a total axial load of 8 psi was applied, (deviator stress  $\sigma_d = 6$  psi). Specimen testing was started at the end of the 600

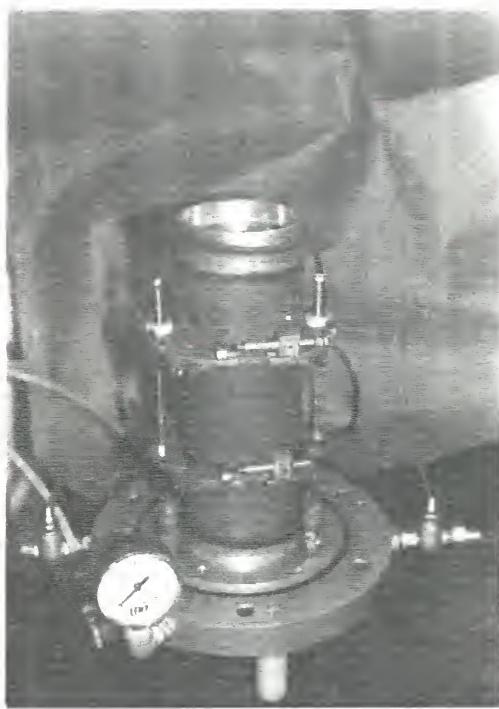


Figure 5.14 LVDT Clamps Placed Over the Specimen

repetitions of conditioning with same stress as the last 200 repetitions. The stress level of  $\sigma_d = 6$  psi and  $\sigma_3 = 2$  psi was considered representative of the stresses encountered in the field by the subgrade layer. Stresses due to the surface load were conservatively calculated using the Boussinesq equations at a depth of 25 inches below the pavement surface. An 18 kip axle loading and 100 psi tire pressure were assumed. The resulting stress increments were  $\Delta\sigma_v = 6.5$  psi and  $\Delta\sigma_h = 0.13$  psi. To these were added geostatic stresses, and the totals rounded off to give 8 psi vertical and 2 psi horizontal. All specimens were tested for 10,000 repetitions. Test data were recorded at or near the following repetition levels; 1, 10, 100, 200, 400, 600, 800, 1,000, then every 1,000 thereon up to the termination of the test. Test data monitored included axial load, axial deformation and radial deformation. These were recorded on a strip chart recorder manufactured by the Gould Brush Company.

## CHAPTER VI RESULTS AND DISCUSSION

### 6.1 Introduction

This chapter discusses the results obtained during the course of the testing program. The principal objective of the test program was to characterize the permanent deformation of the four subgrade soils under a variety of water conditions. All four soils were triaxial repetitive load tested at optimum water contents. In addition, the A-3 and A-2-4 soils were tested under three different water retention conditions, corresponding to three different heights above the water table. These heights and water contents were selected from the respective soil-water retention curves. The A-2-6 and A-5 soils were tested at only one additional water retention condition. In these soils, because of their high capillary fringes, there was no significant change in water content up to a height of 11 feet above the water table level. The selected water retention conditions were chosen, based on the range of economical fill height above the water table. Table 6.1 summarizes the test program conditions.

### 6.2 Deformation Characteristics at Optimum Water Content

The relationships between axial permanent strain,  $\epsilon_p^a$ , and number of stress applications, N, were established for all the project soils. Tables C.1 through C.4 in Appendix C provide the test results. These include the permanent and resilient axial and radial strains, resilient modulus, and resilient Poisson's ratio.

Table 6.1 Test Program of Project Soils Under Varied Water Content

Height Above Water Table in inches	Soil Type			
	A-3	A-2-4	A-2-6	A-5
	Water Content Percentage			
15	13.24			
18	10.53			
24	8.10			
30		11.70		
36		10.50		
48		8.70		
0			12.20	
36				44.40

The resilient modulus was defined as the deviator stress divided by the axial resilient strain, while the resilient Poisson's ratio was defined as the radial resilient strain divided by the axial resilient strain. Figure 6.1 shows an arithmetic plot of the number of stress applications versus the accumulated axial permanent strain for all four project soils. Figures 6.2 through 6.5 provide semilogarithmic plots of the number of stress applications versus both the total and accumulated permanent axial strains, for the A-3, A-2-4, A-2-6, and A-5 soils, respectively.

The A-3 soil was compacted according to Standard Proctor to provide optimum conditions of dry density equal to 103.1pcf and water content equal to 14.8 percent. These were therefore the initial conditions prior to the repetitive load testing. The water mixed with the dry material is essentially free water for a granular free draining material such as the A-3 soil. When the confining pressure was applied to the specimen as part of the conditioning procedure, the specimen drained 113 grams of water. The end of conditioning water content was therefore 10.4 percent. By the end of the actual testing, an additional 54 grams of water had drained, resulting in a final overall water content of 8.6 percent.

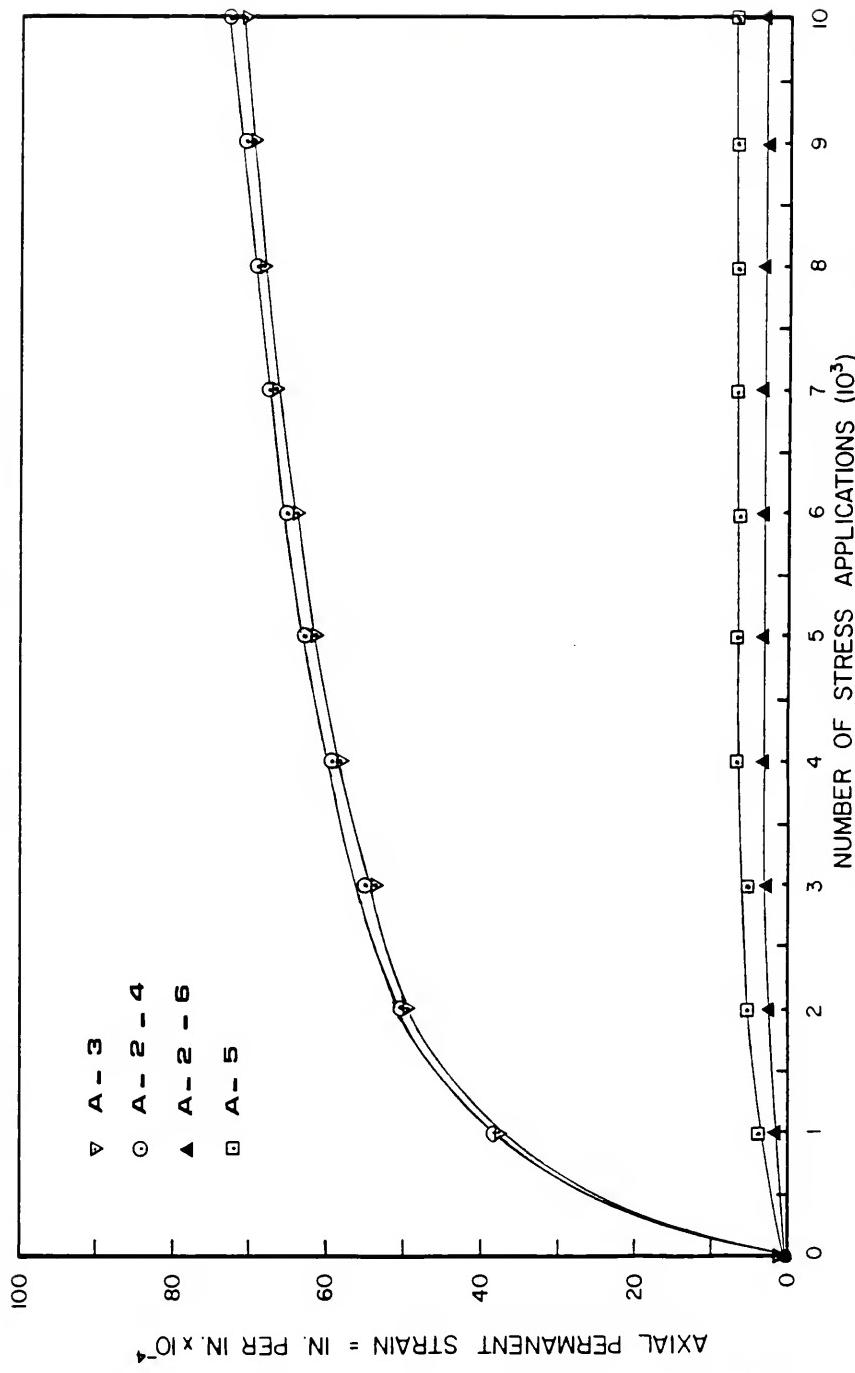


Figure 6.1 Accumulated Axial Permanent Strain Versus Number of Stress Applications for All Project Soils

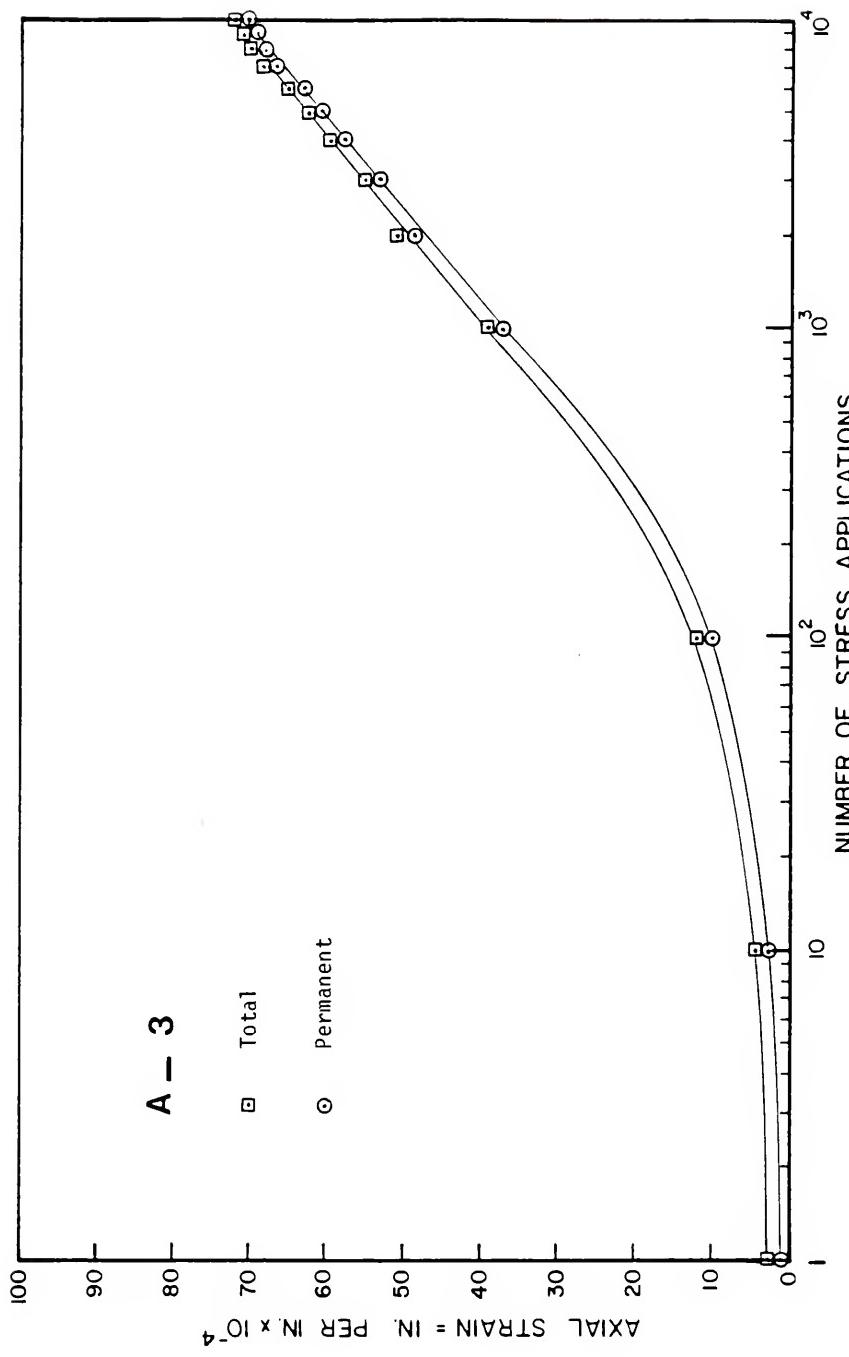


Figure 6.2 Accumulated Axial Strain Versus Number of Stress Applications for A-3 Soil

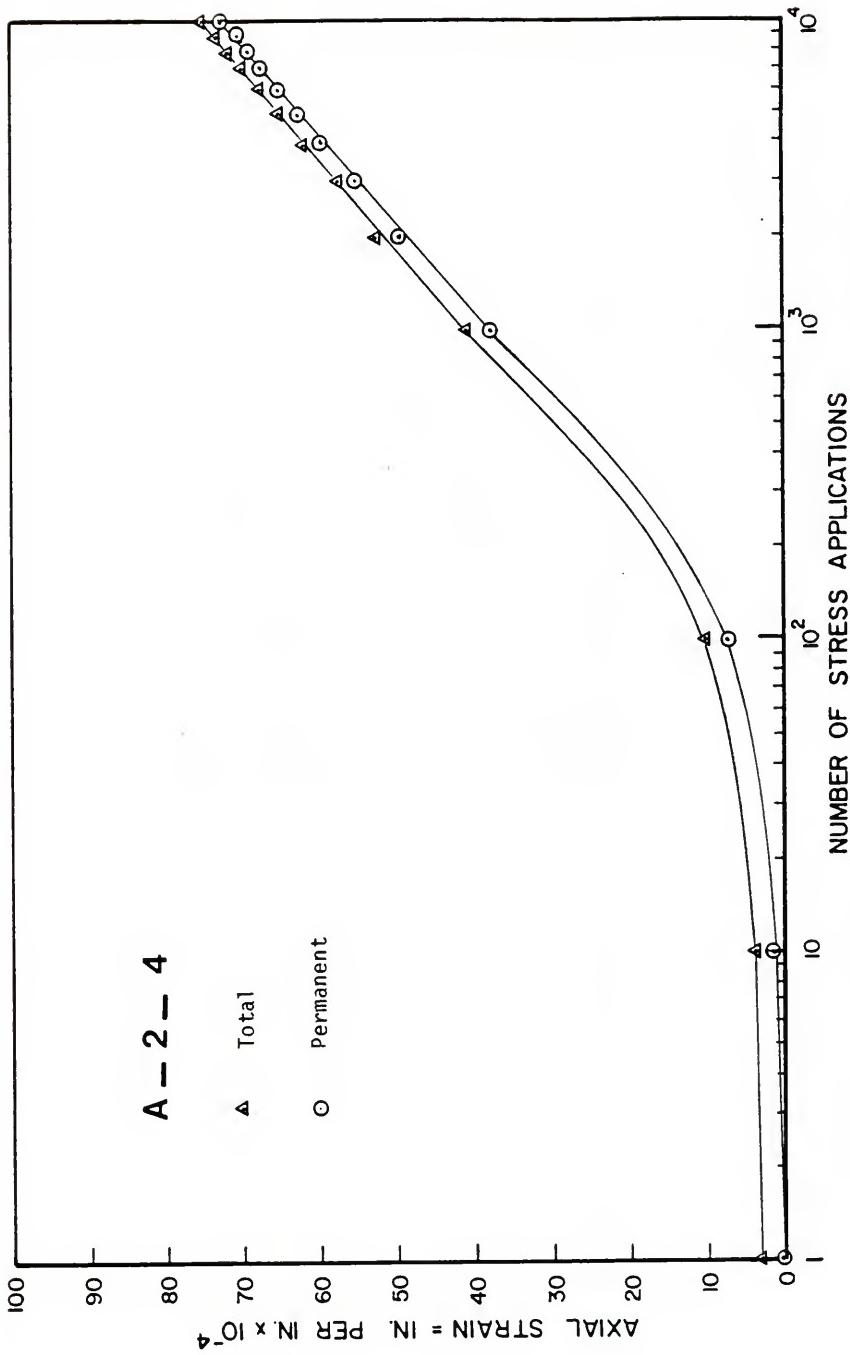


Figure 6.3 Accumulated Axial Strain Versus Number of Stress Applications for A-2-4 Soil

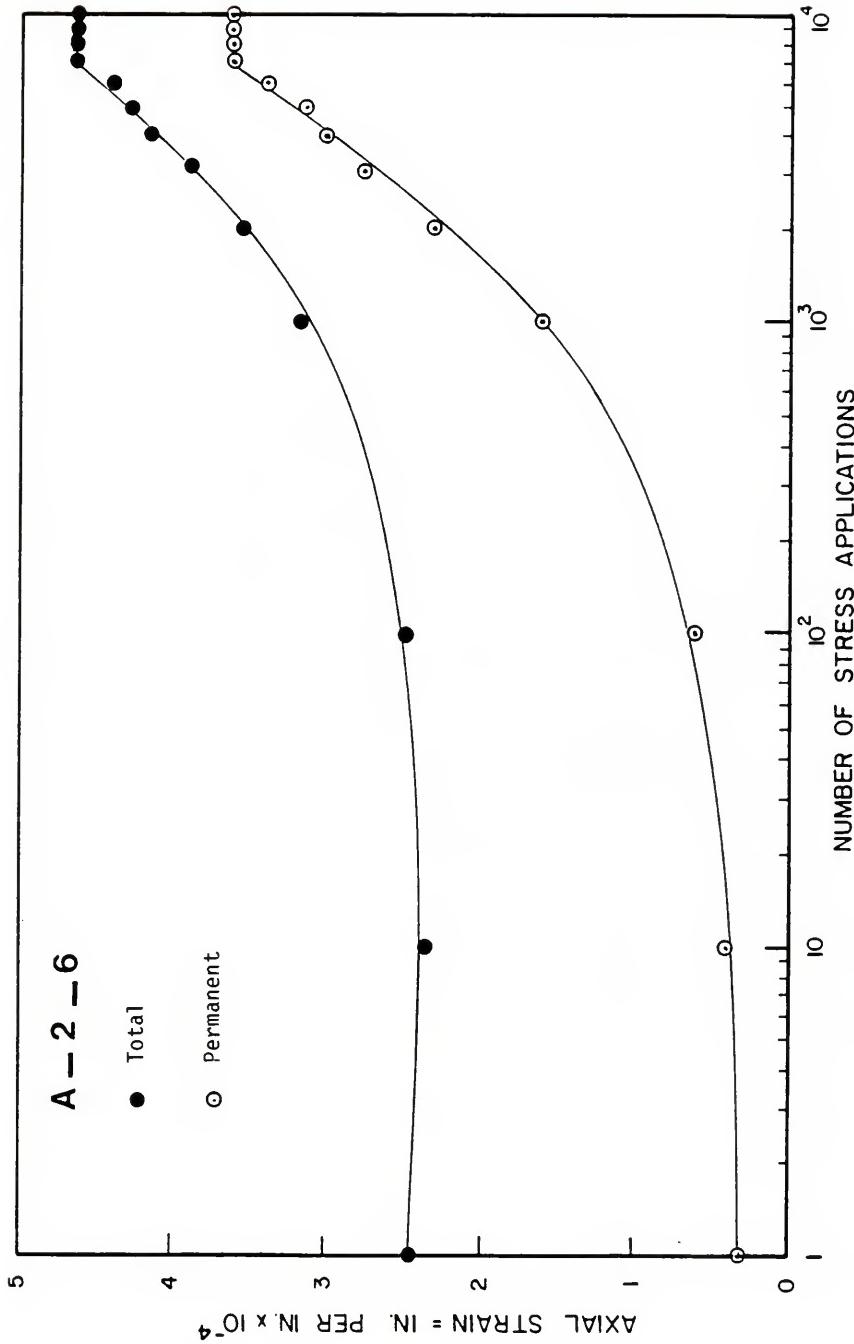


Figure 6.4 Accumulated Axial Strain Versus Number of Stress Applications for A-2-6 Soil

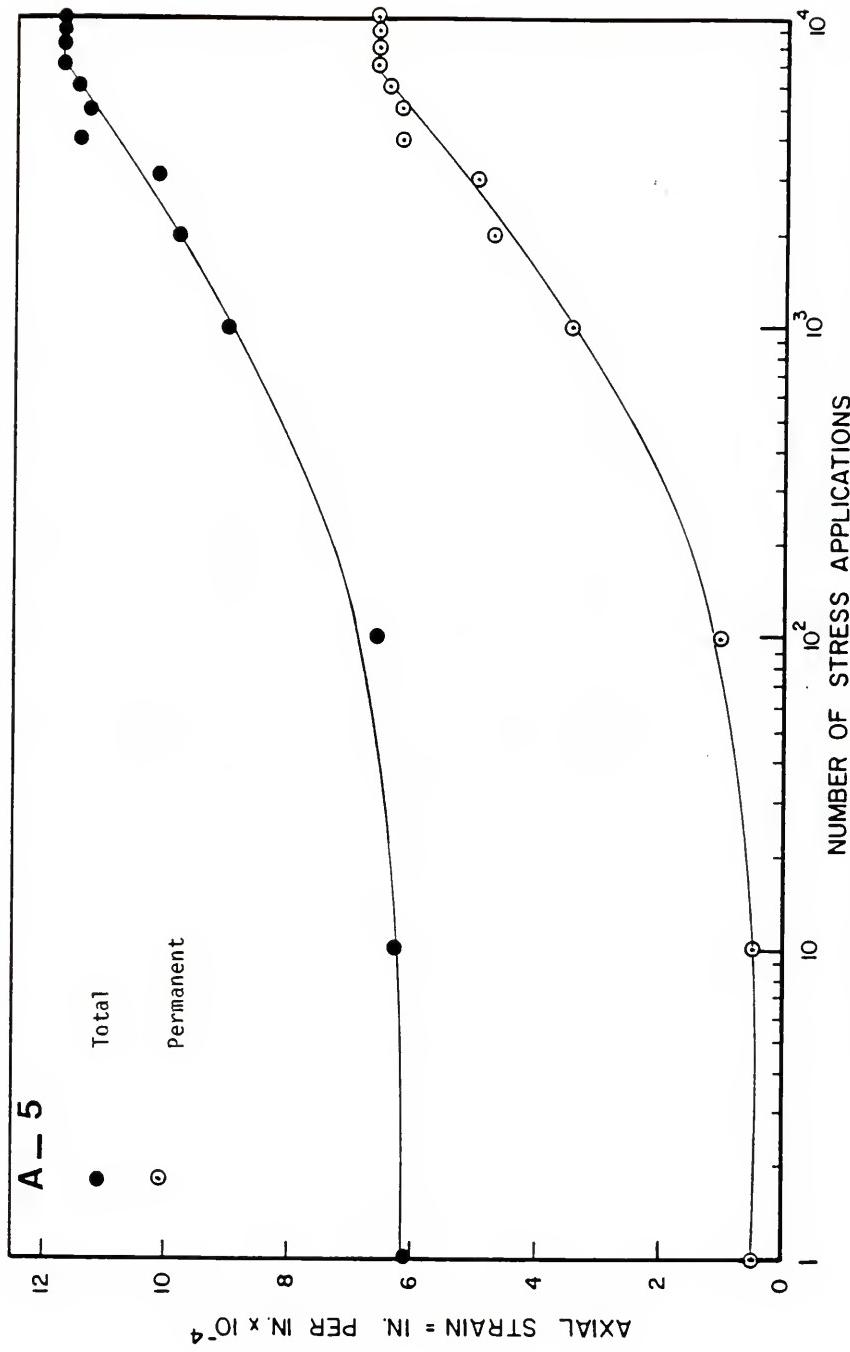


Figure 6.5 Accumulated Axial Strain Versus Number of Stress Applications for A-5 Soil

The plot for the A-3 soil in Figure 6.1, showing the relationship between accumulated axial permanent strain and number of stress applications, is typical of such plots reported in the literature. Two stages can be recognized in this relationship. The first stage shows a high rate of deformation and can be represented by a power function of the form

$$\frac{\epsilon_p^a}{N} = AN^B$$

where

$\frac{\epsilon_p^a}{N}$  = accumulated axial permanent strain

N = number of stress applications

A, B = constants (which would represent intercept and slope terms, respectively, on a log-log plot)

This high rate of deformation then decreases, leading to the second stage where the deformation becomes almost constant and can be represented by a semilogarithmic function of the form

$$\frac{\epsilon_p^a}{N} = A + B \log N$$

Data from both stages were fitted using least squares procedures. There was a transition point between the stages at 1,000 stress applications. Figure 6.2 shows plots of the total accumulated axial strain, which is the sum of the accumulated axial permanent strain and the axial resilient strain, and accumulated axial permanent strain versus number of stress applications. The total and permanent strains followed almost the same trend, indicating the small variation in resilient strain. The range of values for the resilient modulus ( $M_R$ ) was between 24,615 and 43,636 psi with a value of 27,429 psi at 1,000 stress applications. The wide range of values is due to the water draining during the repetitive load testing. The A-3 specimen became stiffer toward the end of the test and the values of  $M_R$  stabilized. It is to be noted that all strain values were read from the strip chart recorder by eye using a magnifying glass,

therefore, they were subject to a small variation considering that each division on the chart represented 0.0005 inch.

Early in the testing program, it was realized that some inconsistencies existed in the measuring of the radial strain. This was believed due to the clamp feet digging into the specimen. Two reasons for this were, the small contact area between the clamp's feet and the specimen, and the elasticity of the rubber bands used to pull the two halves of the LVDT's clamps together. An effort was made to reduce these problems by increasing the width of the feet and using rubber bands with appropriate elasticity. Radial strain measurements, however, were still not satisfactory all of the time. This resulted in resilient Poisson's ratios being higher than 0.5 in some cases. In all tests, physical inspection was made to assure that there was no indication of the feet digging into the specimen. It was finally decided that only the axial strain could be considered in this study. Similar testing problems have been reported by Monismith et al., 1975, and by Brabston, 1982.

The A-2-4 soil was compacted according to Standard Proctor with a dry density of 107.2pcf and a water content of 11.8 percent. In this soil there was no drainage of water out of the specimen during the entire test. This was to be expected because of the fine gradation and the presence of 11 percent Fines (passing #200 sieve). Figure 6.1 shows the almost identical axial permanent strain - number of stress applications relationships of the A-2-4 and A-3 soils. The  $M_R$  values for the A-2-4 soil ranged between 19,750 and 25,622 psi with a value of 24,947 psi at 1,000 stress applications. These values are a consequence of the higher resilient strains, which can be observed by comparing Figures 6.2 and 6.3.

The A-2-6 soil was also compacted according to Standard Proctor and resulted in a dry density of 120.3pcf and a water content of 11.6 percent. No water drained from the specimen during the entire test. The A-2-6 plot in Figure 6.1 shows two distinct stages. The first stage could be represented by a power function as before, except that there existed a very shallow slope at the start up to 1,000 stress applications. This then increased from that point up to 7,000 stress application. In the second stage, between 7,000 - 10,000 applications, the specimen response was a plateau with no increase in the axial permanent strain. Figure 6.4, at a much enlarged scale, shows that in the beginning of the test the axial resilient strain was much larger than the axial permanent strain. However, by 1,000 stress applications, almost equal values were obtained. The resilient strain continued to decrease until it reached a constant value between 7,000 and 10,000 applications. This A-2-6 soil is customarily used as a base course in Chipley, West Florida, compacted to Modified Proctor. It is not therefore surprising it performed so well even when compacted only to Standard Proctor. The well graded distribution of the grain sizes results in high dry density and low void ratio, which in turn contribute to less permanent deformation. The  $M_R$  values ranged between 28,235 and 60,000 psi with a value of 38,400 psi at 1,000 stress applications. The reason for the wide range of  $M_R$  values could be attributed to re-orientation of particles from the beginning of the test up to 5,000 repetitions. The  $M_R$  values from 6,000 to 10,000 repetitions were constant, as if the specimen had reached a stage of threshold. The  $M_R$  value of 60,000 psi is attributed to the low stress values used in the repetitive loading compared to the peak deviator stress at static

triaxial failure. This ratio was approximately 0.25 (repetitive deviator stress/peak deviator stress at failure,  $\sigma/2\sigma_u$ , see Table 4.6). Both terms in the above ratio are based on confining pressures,  $\sigma_3$ , of 2 psi.

The reason for the wide range of  $M_R$  values is that the stress level applied is too small for such material. It appears that it will take larger number of repetitions to reach a constant value. Again, the values between 3000 and 6000 repetitions were read off the strip chart and in this range it was very difficult to precisely estimate the difference in resilient strain values.

The A-5 soil was compacted to Standard Proctor at a dry density of 72.0pcf and a water content of 37.84 percent. This soil showed similar behavior to the A-2-6 soil, except that the first stage had a much steeper slope, as shown in Figure 6.1. Figure 6.5 shows that initially the axial resilient strain was much higher than the axial permanent strain. By 3,000 stress applications, equal values were observed. A plateau was then reached between 7,000 and 10,000 applications.  $M_R$  values ranged between 10,213 and 12,000 psi, with a value of 10,909 psi at 1,000 stress applications.

Table 6.2 summarizes the prediction equations, their coefficients, and the R square ( $R^2$ ) values for the four project soils tested at optimum moisture content conditions.

### 5.3 Deformation Characteristics at Varied Water Retention Conditions

To determine the soil water-pressure conditions for the second phase repetitive triaxial tests, it was necessary to model the water retention conditions as they would exist in the subgrade soil, in-service, at equilibrium with the designated water table. To accomplish this, two

Table 6.2 Summary of the Prediction Equations and Their Coefficients for All Project Soils at Optimum

Soil Type	Number of Stress Applications	Type of Function	Intercept A	Slope R	R <sup>2</sup>
A-3	1 - 1000	Power	0.97	0.53	0.99
	1000 - 10,000	Semi log	-62.14	33.29	1.00
A-2-4	1 - 1000	Power	0.14	0.87	1.00
	1000 - 10,000	Semi log	-62.29	33.53	1.00
A-2-6	1 - 7,000	Power	0.20	0.31	0.93
	7,000 - 10,000	Linear-flat	3.60	0	1.00
A-5	1 - 7,000	Power	0.22	0.38	0.92
	7,000 - 10,000	Linear-flat	6.6	0	1.00

studies were made, one using soil columns and the second using Tempe pressure cells. The soil columns, described in Chapter 5, represent a one-to-one simulation of the pavement substructure. Subgrade, stabilized subgrade and even the base course can be prepared as they would occur in the field. A profile of water content and soil suction can then be found experimentally for a particular pavement subsurface. Soil columns up to 4 feet in height have been employed. The Tempe pressure cells on the other hand provide a single point value, and have been used only for the subgrade materials. Since small volumes of soil are used, these tests are much more rapid than column testing. Conditions representing up to 11 feet of subgrade above a water table have been modeled. Results from the two procedures compare reasonably well, as shown in Figure 6.6 for the A-3 soil.

Testing with the Tempe cell had the following advantages:

1. Obtaining full saturation was much easier and faster in the Tempe cell than in the column experiments.
2. Reaching equilibrium was also much faster due to the smaller size specimen.
3. The water content obtained from the Tempe cell was representative of the entire specimen. That obtained from the column study was representative only of a localized area around the tensiometer.
4. Due to its small size, the Tempe cell does not experience any effect due to temperature. This could pose a problem in the 4-foot column. Temperature variations leading to a scattering of test results have been reported by Spangler and Handy (1982).
5. A large number of soils can be studied in a smaller space and in a shorter time.
6. Specimens can be retrieved from actual pavement profiles and studied using the Tempe cells.

The testing of the project soils, under varied water retention conditions, was limited to a range between 1 and 4 feet above the water table.

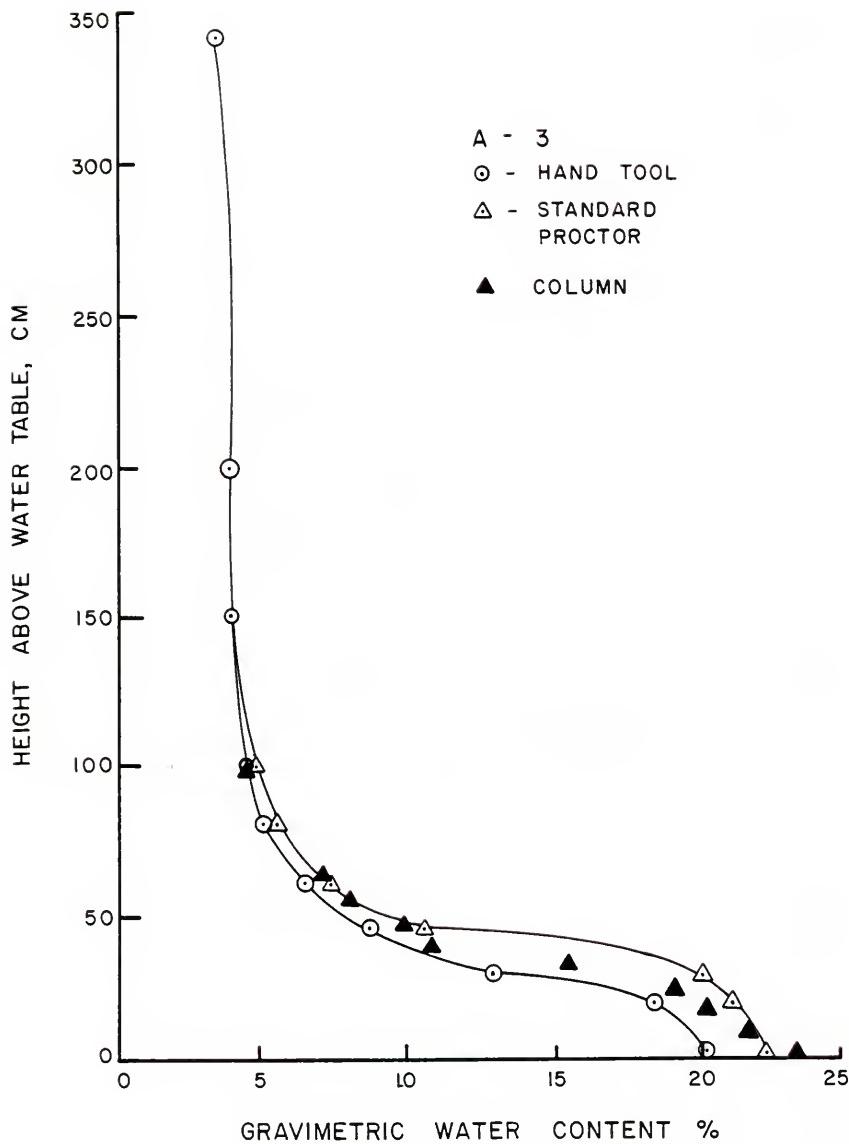


Figure 6.6 Gravimetric Water Content Versus Height Above Water Table for A-3 Soil From Both Tempe Cells and Column Study

The A-3 soil was tested by repetitive loading under three different water retention conditions. These had water content values of 13.50, 10.53, and 8.10 percent representing 15, 18, and 24 inches above the water table, respectively. Tables C.5 through C.7 contain the deformation characteristics data collected from these tests, respectively. Figure 6.7 shows a cumulative plot of axial permanent strain  $\epsilon_p^a$  versus number of stress applications N for the three water retention conditions. The curves stack, with the one representing 24 inches at the bottom, that representing 18 inches in the middle and, the one representing 15 inches at the top. For a certain number of stress applications, the bottom curve yields the lowest axial permanent deformation and the top curve the highest. A power function, as presented before, was found to fit all the data in all three tests.

The A-2-4 soil was also tested under three different levels of water retention. Water contents chosen were 11.70, 10.50, and 8.7 percent, representing 30, 36, and 48 inches above the water table. Tables C.8 through C.10 contain the deformation characteristics data collected from these tests, respectively. Figure 6.8 provides the plots of  $\epsilon_p^a$  versus N. Again, as expected, the plot representing the greatest distance, 48 inches, was located at the bottom of the figure, that representing 36 inches was in the middle and the 30 inches plot was on top. The A-2-4 soil showed behavior very similar to that of the A-3 soil. Power functions again were used to fit the data.

#### 6.4 Deformation Characteristics at Selected Water Retention Conditions

The A-2-6 and A-5 soils were each tested at only one selected water retention level. The A-2-6 specimen was prepared by soaking in water for

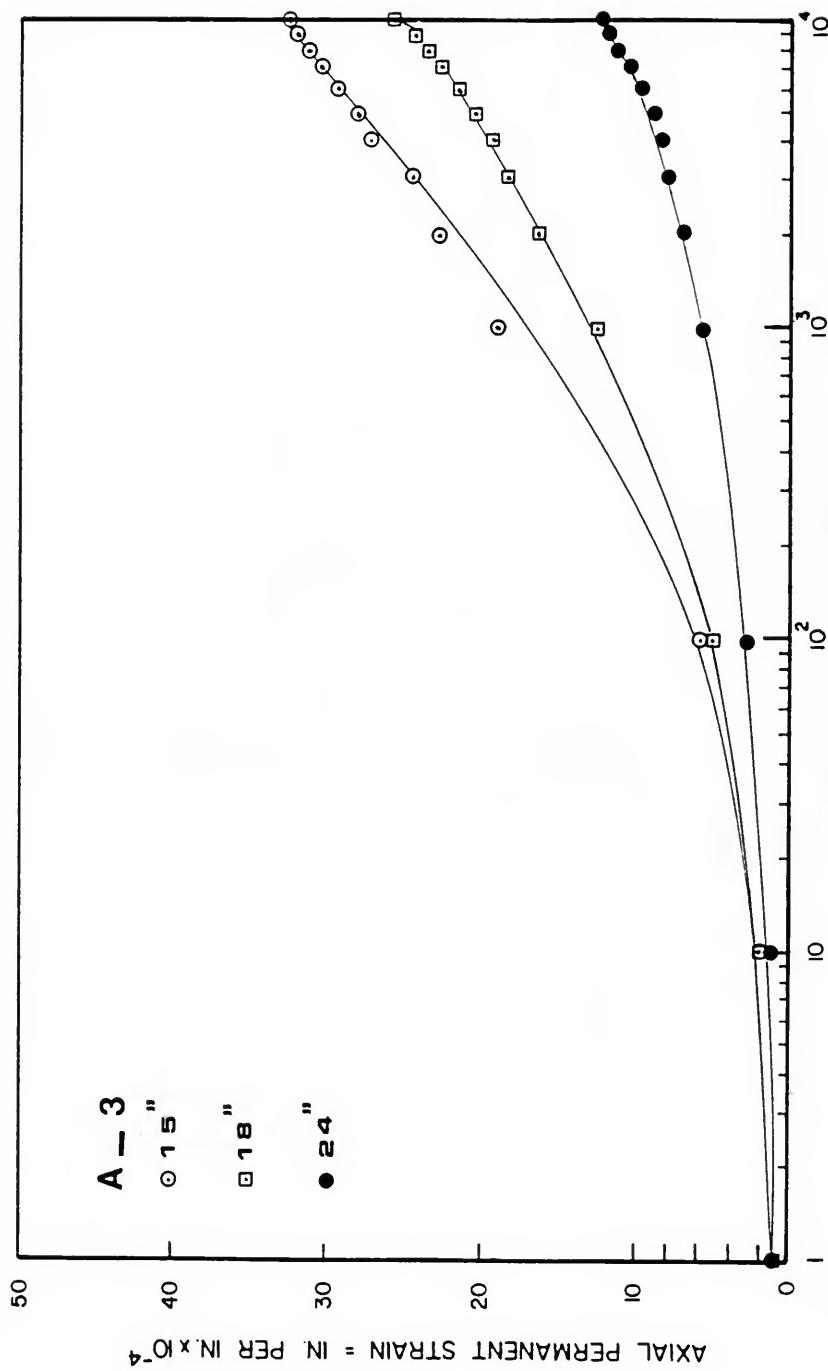


Figure 6.7 Accumulated Axial Permanent Strain Versus Number of Stress Applications for A-3 Soil at Varied Water Retention Conditions

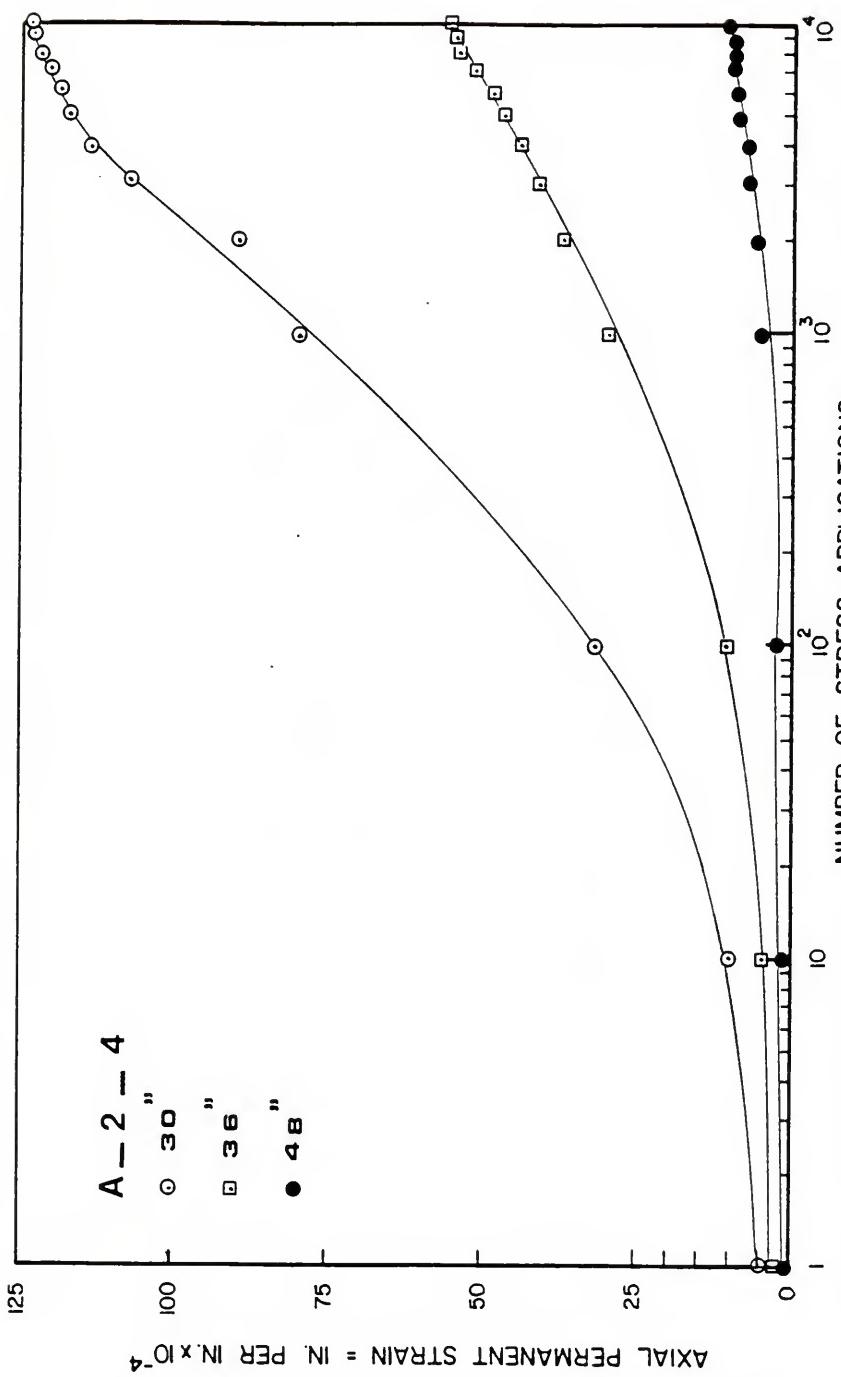


Figure 6.8 Accumulated Axial Permanent Strain Versus Number of Stress Applications for A-2-4 Soil at Varied Water Retention Conditions

8 days. This procedure led to a water content of only 12.2 percent. This was considered as saturated as this specimen could be. The specimen was never placed under any capillary tension. The condition therefore represents the worst that might be encountered in the field.

The A-5 soil was placed under a capillary tension to represent a height of 36 inches above the water table. Its water content was 44.4 percent. Tables C.11 and C.12 contain the deformation characteristics data for the A-2-6 and A-5 soils, respectively. Figures 6.9 and 6.10 show the plots of  $\epsilon_p^d$  versus N. A power function was found to fit the data in the first stages up to 7,000 and 5,000 applications for the A-2-6 and A-5, respectively. The second stage for both soils was a plateau where no permanent strain increase was observed.

Table 6.3 includes a summary of  $M_R$  values, the correlation equations and their coefficients, and the  $R^2$  values for all four project soils, at their different water retention conditions.

### 6.5 Deformation Characteristics During Dynamic Conditioning

The dynamic conditioning at the start of each cyclic test consisted of three stress levels, each applied for 200 repetitions. Only results from the optimum water condition tests are discussed in this section. Trends in all soils were the same. The highest permanent strain resulted from applying the highest stress level. Again, the A-3 and A-2-4 soils had similar characteristics and the A-2-6 and A-5 soils had similar characteristics.

Figure 6.11 shows plots of axial permanent strain versus the number of stress applications at three stress levels for the A-3 specimen compacted at optimum water content. Table C.13 summarizes the data

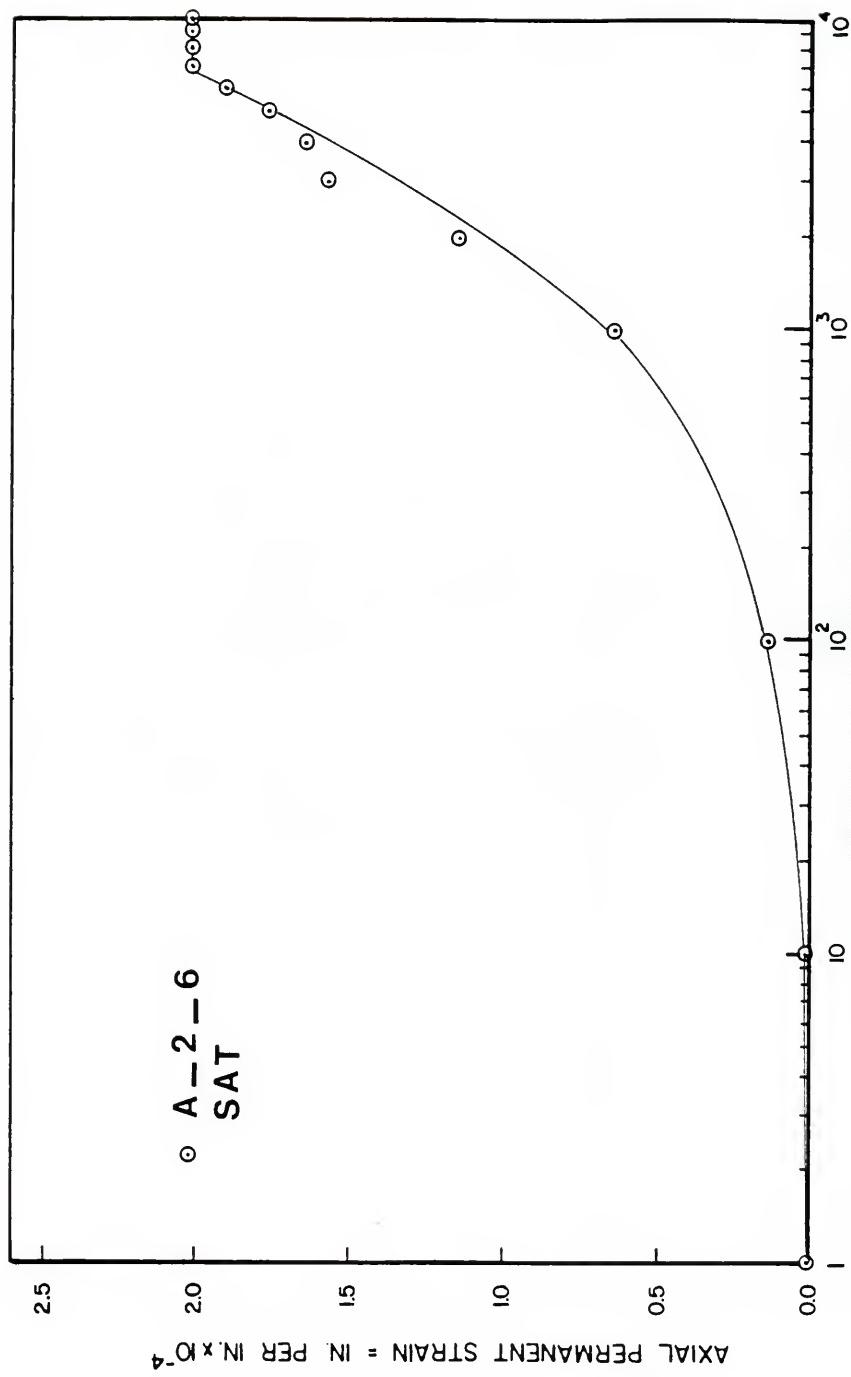


Figure 6.9 Accumulated Axial Permanent Strain Versus Number of Stress Applications for A-2-6 Soil at Varied Water Retention Conditions

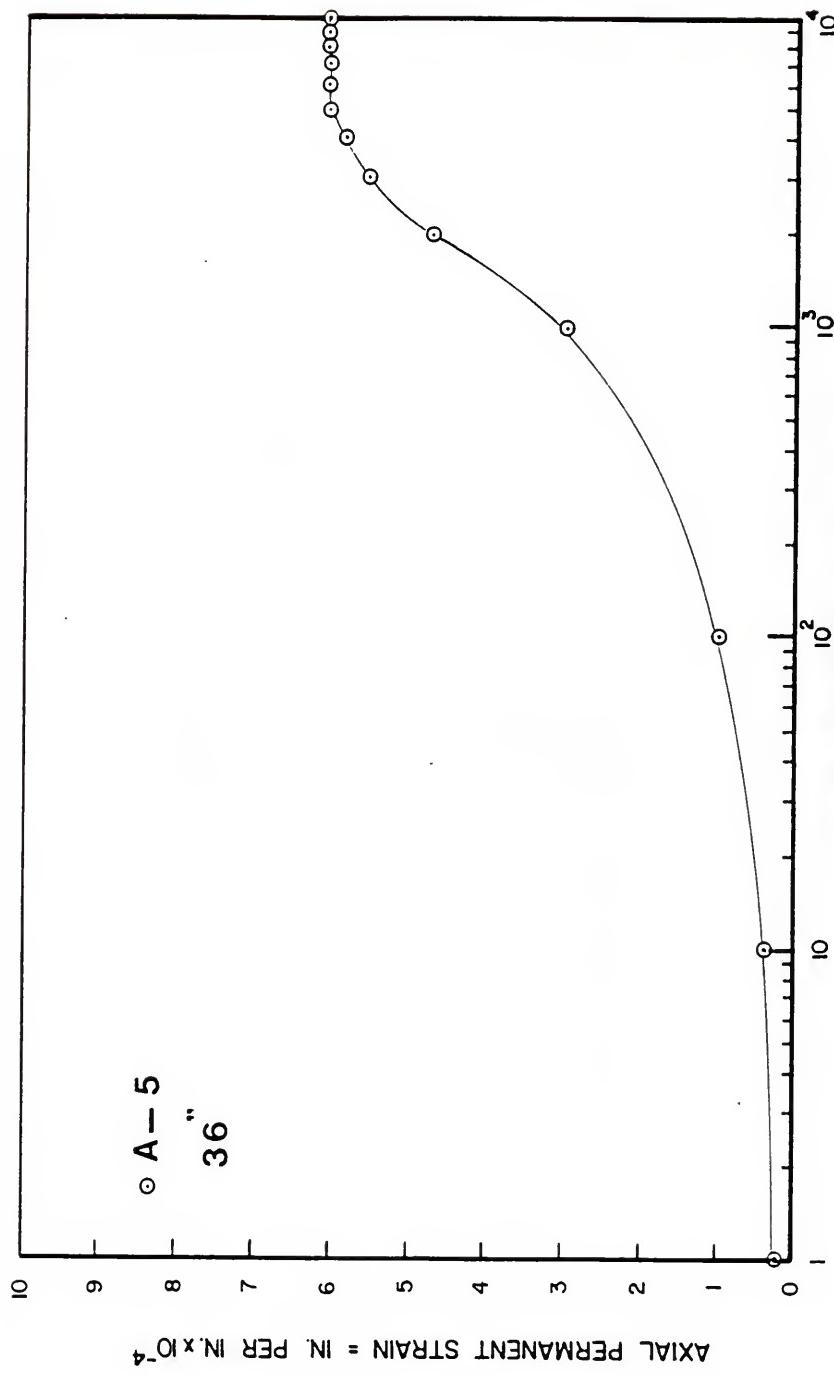
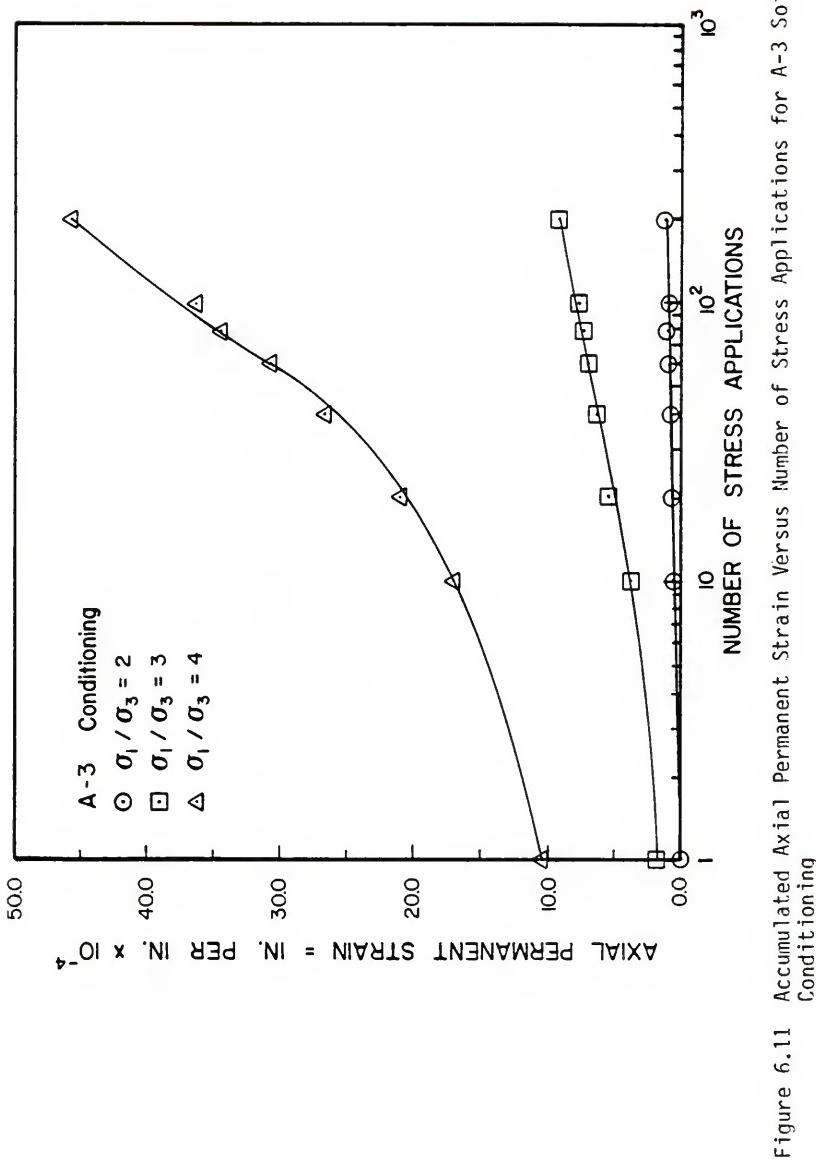


Figure 6.10 Axial Permanent Strain Versus Number of Stress Applications for A-5 Soil at Varied Water Retention Conditions

Table 6.3 Summary of Prediction Equations and  $M_R$  Values for All Project Soils at Different Water Retention Conditions

Soil Type	Water Retention Condition %	Range of $M_R$ psi	$M_R$ at $N = 1,000$ psi	Number of Stress Applications N	Type of Function	Intercept A	Slope B	$R^2$
A-3	13.50	27,429-35,556	32,000	1-10,000	power	0.89	0.41	0.98
	10.53	33,103-56,471	35,553	1-10,000	power	0.90	0.37	1.00
	8.10	36,923-43,636	38,400	1-10,000	power	0.72	0.30	1.00
A-2-4	11.70	23,577-37,723	27,738	1-10,000	power	5.18	0.37	0.97
	10.50	26,667-35,556	28,235	1-10,000	power	2.18	0.36	0.99
	8.70	24,000-32,000	30,000	1-10,000	power	0.12	0.49	0.98
A-2-6	12.20	43,478-48485	43,478	1- 7,000	power	$5.43 \times 10^{-3}$	0.68	0.97
	48,000	--	--	7,000-10,000	linear-flat	2.00	0	1.00
	10,101-11,173	10,118	--	1-5,000	power	0.16	0.43	0.99
A-5	44.40	--	--	5,000-10,000	linear-flat	6.06	0	1.00
	11,152	--	--	--	--	--	--	--



obtained during the dynamic conditioning. It contains values of the axial permanent strain, the resilient strain and the resilient modulus.

The first stress level was  $\sigma_1/\sigma_3 = 2$  ( $\sigma_1 = 4$ ,  $\sigma_3 = 2$  psi). At this stress level the resilient (recoverable strain) was not detected by the equipment used. The axial permanent strain showed an increasing trend with a very shallow slope.

The second stress level was  $\sigma_1/\sigma_3 = 3$  ( $\sigma_1 = 6$ ,  $\sigma_3 = 2$ ). The resilient strain remained constant up to 80 repetitions then increased. The axial permanent strain showed a high rate of increase at the start, up to 40 repetitions, then a constant rate developed.

The third stress level was  $\sigma_1/\sigma_3 = 4$  ( $\sigma_1 = 8$ ,  $\sigma_3 = 2$  psi). The resilient strain fluctuated but within a small range. The axial permanent strain showed a continuous increase with a much higher rate at the lower stress levels. Such behavior was also reported by Monismith et al. (1975). A power function was found to fit the relationships between  $\epsilon_p^a$  and N.

Figure 6.12 shows plots of axial permanent strain versus the number of stress applications at three stress levels for the A-2-4 specimen compacted at optimum water content. Table C.14 summarizes the data obtained. The A-2-4 specimen showed similar behavior to that of the A-3 specimen, except that for the first stress level ( $\sigma_1/\sigma_3 = 2$ ) the resilient strain was detectable and remained constant. It is to be noted that resilient strain values for the A-2-4 specimen were much larger than the A-3 specimen indicating the high tendency for rebound.

Figure 6.13 shows plots of axial permanent strain versus the number of stress applications at three stress levels for the A-2-6 specimen compacted at optimum water content. Table C.15 summarizes the data

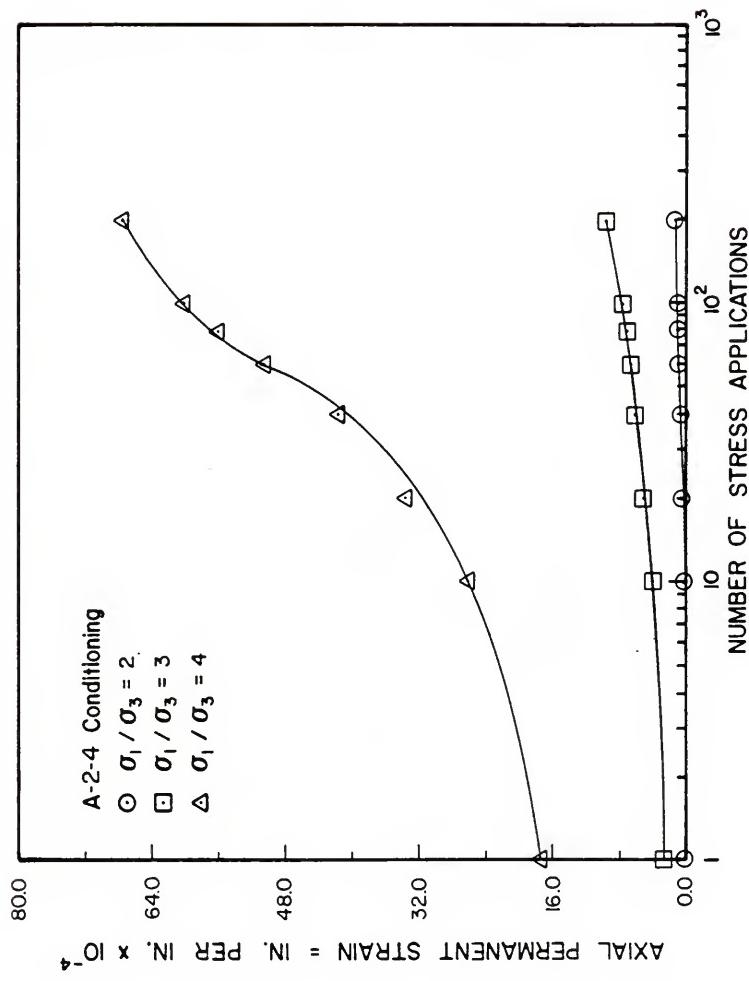


Figure 6.12 Axial Permanent Strain Versus Number of Stress Applications for A-2-4 Soil During Conditioning

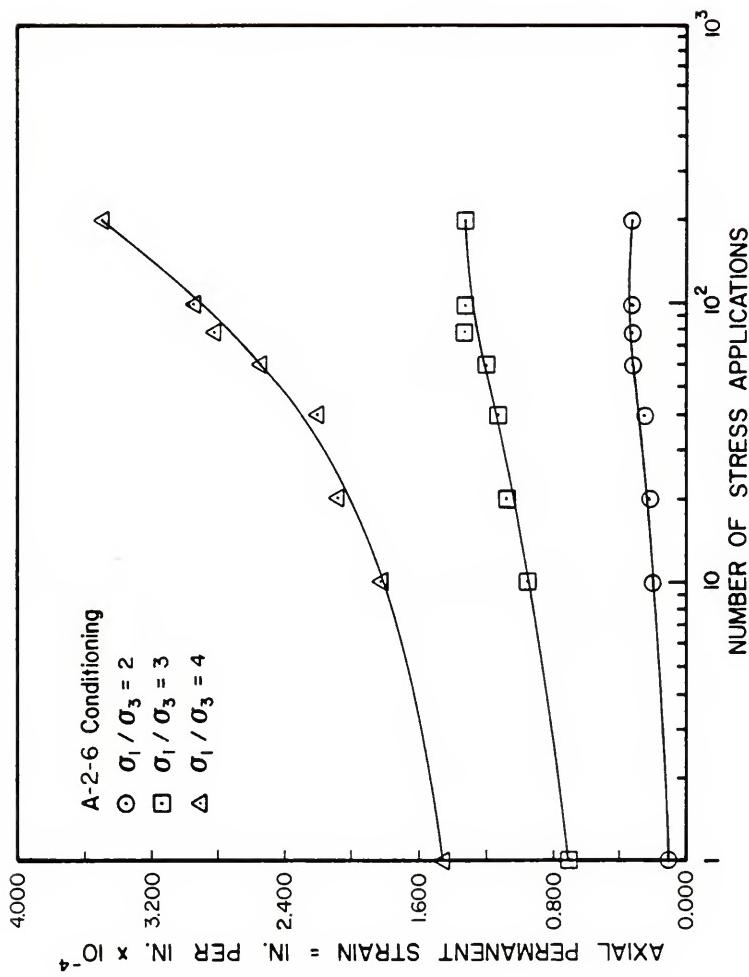


Figure 6.13 Axial Permanent Strain Versus Number of Stress Applications for A-2-6 Soil during Conditioning

obtained. At the first stress level the resilient strain was too small to be detected. The axial permanent strain was relatively small but could be detected. The  $\epsilon_p^a$  showed an increase up to 60 repetitions then leveled off. This was a consequence of the low stress level.

At the second stress level, resilient strain was detectable and showed some fluctuation within a small range. The axial permanent strain again showed an increase up to 80 repetitions then leveled off.

At the third stress level the axial permanent strain showed two distinct stages. In the first there is a high rate of deformation up to 40 repetitions then a second stage with constant rate.

Figure 6.14 shows plots of axial permanent strain versus the number of stress applications at three stress levels for the A-5 specimen compacted at optimum water content. Table C.16 summarizes the data obtained. At the first stress level the resilient strain showed a relatively high value compared to the other three specimens. This indicates the high rebound characteristic of the A-5 soil.

The axial permanent strain increased rather rapidly at the start of the first stress level then became constant. The second stress level showed similar behavior to that in the first stress level except that the axial permanent strain almost leveled off between 80 and 200 repetitions. The third stress level showed a continuous increase in the axial permanent strain all the way to the end of the 200 repetitions. The resilient strain fluctuated within a small range but the actual values were very high and the specimen rebound could be seen by the naked eye during the conditioning stage.

From the above it can be seen that different characteristics were possessed by the different soils. These resulted from such soil

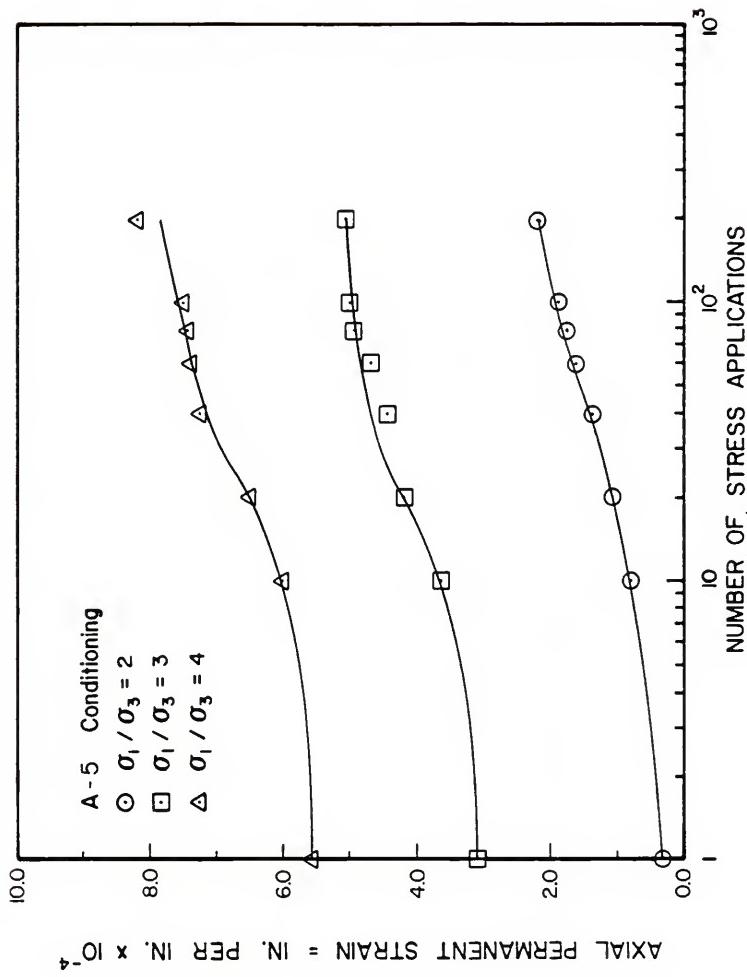


Figure 6.14 Axial Permanent Strain Versus Number of Stress Applications for A-5 Soil During Conditioning

properties as high permeability for the A-3 soil, the presence of 11 percent fines in the A-2-4 soil, the good gradation and higher density of the A-2-6, and the very high percent of fines (94 percent) in the A-5 soil. In general it can be stated that, for granular materials (i.e. A-3 and A-2-4), when the deviator stress ( $\sigma_d = \sigma_1 - \sigma_3$ ) increases the resilient modulus either consistently decreases or decreases to a minimum value then increases again. For cohesive soils, the effect of the deviator stress is very pronounced because of the large variation in the resilient strain (as seen in the A-5 soil). Increasing the deviator stress led to a decrease in the resilient modulus to a minimum value, then a slight increase. Behavior similar to that described above has been reported by Morgan (1966) and Seed et al. (1962).

#### 6.6 Application of Test Results

The results obtained from both the engineering properties tests and the repeated load tests can be utilized as follows:

1. They will provide a data base for the essential engineering and physical properties of four common subgrade fill materials available in the State of Florida. This should help in making an economical decision concerning the selection of fill materials and the certification of borrow pits.
2. The soil-water characteristic curves provide a realistic simulation of the water retention in a subgrade. Similar distributions have been found in field investigations reported by Kersten (1944), Russam (1970), and Janssen and Dempsey (1980). Rather than assuming full saturation, it is appropriate to estimate subgrade strength based on such soil-water

characteristic curves. This should be superior to simply assuming a regional (environmental) factor (1 for Florida) as part of the AASHTO design procedures. From the soil-water characteristic curves it can be seen that for a subgrade fill at a height which falls within the capillary fringe (approximately fully saturated) the water will have a detrimental effect on the strength. This has been reported by Barber and Sawyer (1952). The minimum height of subgrade fill should therefore be greater than the height of the capillary fringe. Such a fill benefits from the strengthened effect due to capillary suction and the negative pore water pressures. This was demonstrated in the repetitive load testing. Figures 6.7 and 6.8 show that the greatest axial permanent deformations occurred in the specimen representing the top of the capillary fringe, 15 inches in the A-3 soil and 30 inches in the A-2-4 soil. For the specimens representing heights above the capillary fringe, the axial permanent strains are greatly reduced.

For the project soils, the A-3 subgrade fill could be built to a minimum height of 15 inches. This is based on an average value from Figure 4.9. Any greater height provides additional assurance for safe subgrade performance. This was demonstrated in the repetitive load testing, where the permanent deformation decreased as the height above 15 inches increases (see Figure 6.7). Since the difference between 15 and 24 inches is small, it is probably advisable to build A-3 subgrade fills up to 24 inches. This is in agreement with recommendations made by Wright and Paquette, 1979.

For the A-2-4 soil, the minimum height is 30 inches. A height of 36 inches is recommended. This height is supported by the repetitive load testing results shown in Figure 6.8. A 48-inch height would not be justified.

For the A-2-6 soil, there is no distinguishing capillary fringe. Figure 4.11 shows a gradual uniform decrease in water content with height above the water table. The significant property of this soil is its well-graded nature and consequently its low void ratio. Because of this, the structure is strong even when the soil has been submerged under water for one week. For the A-2-6 soil, a 12-inch height is suggested. This would represent a substantial savings compared to using the 24 inches of the current FDOT guidelines. Figure 6.9 shows the very low permanent deformation and high resilient modulus of the A-2-6 soil.

The shape of the soil-water characteristic curve for the A-5 soil is dependent on the method of compaction and the voids formation, as shown in Figure 4.12. It is therefore difficult to assign a conclusive capillary fringe height. In both cases the soil remained near saturation up to 200 cm (6.5 feet) with the curves showing a break at 91 cm (3 feet). The water content loss at this break was approximately 2 percent. A third specimen was saturated and placed under a 91-cm (3-foot) negative column. The water content obtained was 44.40 percent. This specimen was then tested under repetitive loading. Its resilient strain was very high compared to the other three soils. In addition, swelling was observed at the end of the

saturation stage. Such serious drawbacks must be very carefully weighed against economics before deciding to use the A-5 soil as subgrade fill. However, it is recognized that in some circumstances it will be the only material available within economical hauling distance. In these cases a height between 4 and 6 feet (5 feet average) is recommended as a reasonable fill height.

The recommendations in this section pertain only to the soils tested in this project. The capillary fringes reported were based on the draining case (retention condition) and a soil structure achieved by the mechanical compaction method. Figure 6.15 shows the recommended heights of subgrade fill for the project soils, based on the retention case capillary fringes.

3. The resilient modulus, obtained from the laboratory testing of the project soils under a variety of water retention conditions, can be used as input in a multilayer elastic analysis. BISAR (Bitumen Structures Analysis in Roads) is a general purpose program for computing stresses, strains and displacements in elastic multilayered systems subjected to a uniform load over a circular surface area. BISAR was introduced in 1972 by Koninklijke/Shell Laboratorium, Amsterdam (KSLA). Obtaining the stresses and strains from such an analysis will allow a comparison to be made with Shell Oil Company Criteria (Shell Criteria) for limiting subgrade strain.

The following parameters were used in a series of BISAR program runs:

-Wheel load = 9,000 lbs. (18,000-lb. axle load)

-Tire pressure = 100 psi

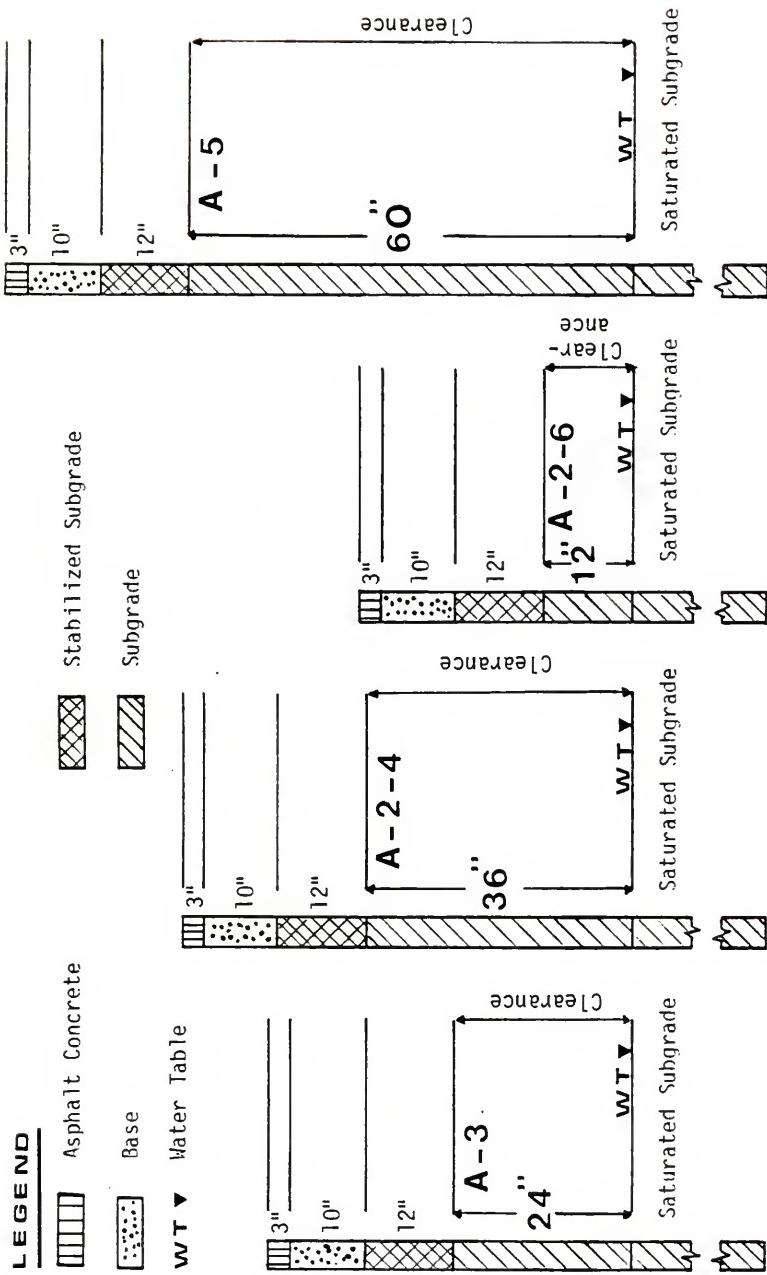


Figure 6.15 Recommended Subgrade Clearance Heights for the Project Soils Based on Capillary Fringe (Retention Case)

$$\text{-Radius of circular surface area} = \left( \frac{9,000}{\pi \times 100} \right)^{1/2} = 5.35 \text{ inches}$$

-Poisson's ratio = 0.35 (for elastic analysis)

Typical values of resilient moduli for the asphalt concrete, lime rock base and stabilized subgrade were obtained from FDOT Material Characterization Laboratory. (Personal Communication with Mr. Ed Leitner FDOT, 1985)

$$\text{-}M_R, \text{ asphalt concrete (3 inches)} = 171,000 \text{ psi}$$

This is type 1 asphalt concrete at 107° F air temperature above the pavement.

$$\text{-}M_R, \text{ lime rock base (10 inches)} = 84,000 \text{ psi}$$

$$\text{-}M_R, \text{ stabilized subgrade (12 inches)} = 60,000 \text{ psi}$$

The stabilized subgrade as used in this research was a mix of three parts subgrade and one part lime rock. The value of 60,000 psi was assumed, based on the values obtained for the subgrade alone (subgrade  $M_R = 48,000$  psi for A-2-6). It would be expected that the stabilized subgrade would possess a higher  $M_R$  value than the subgrade alone.

Resilient moduli for the subgrade layers depend on the material and on its location above the water table. Table 6.4 gives values for the four project soils at 10,000 repetitions, and the corresponding heights above the water table.

The final layer, below the water table, was given a modulus value of 10,000 psi. This was chosen since the weakest material at saturation, the A-5, had an  $M_R$  value of 11,152 psi.

The above parameters were also used as input in BISAR, with an overload 30 kip axle load and 125 psi tire pressure (radius = 6.18 inches). Table 6.5 shows a summary of the strains at the

Table 6.4 Subgrade Resilient Moduli Used as Input in BISAR Programs

Height Above Water Table in inches	Soil Type			
	A-3	A-2-4	A-2-6	A-5
	$M_R$ (Resilient Modulus, psi) at 10,000 Rep.			
15	28,235			
18	34,286			
24	40,000			
30		24,666		
36		30,968		
48		32,000		
0			48,000	
36				11,152

top of the subgrade from laboratory tests and from BISAR's output at 18 and 30 kips. Table 6.6 shows the Shell Criteria, i.e., limiting subgrade compressive strain values corresponding to different numbers of load applications. This Criteria was established by the Shell Oil Company in 1962 and was documented by Dorman and Metcalf (1965). It is still in use today. (Personal communications with Dr. M. W. Witczak of the University of Maryland and Dr. Y. T. Chou of the Corps of Engineers, Waterways Experiment Station at Vicksburg, Mississippi, 1986). The Shell Criteria were developed from elastic analyses of pavements designed according to the California Bearing Ratio (CBR) procedure, and from pavements in the AASHTO Road Test. The Shell Criteria insure that permanent deformations in the subgrade will not lead to excessive rutting at the pavement surface. Considering actual performance results from the AASHTO Road Test in terms of rut-depth, the Shell Criteria in Table 6.6 may be thought to be associated with an ultimate rut depth of the order of 3/4 inch.

The strain corresponding to  $10^3$  load applications in the Shell Criteria was selected for comparison with the strains obtained from the

Table 6.5 Summary of Strains at Top of Subgrade from Laboratory Tests and from BISAR's Output  
at 18 and 30 kips

Height Above Water Table in inches	Soil Type					
	A-3		A-2-4		A-2-6	
	Lab.	18k	30k	Lab.	18k	30k
15	2.13	1.55	2.54			
18	1.75	1.38	2.25			
24	1.50	1.25	2.05			
30				2.43	1.70	2.83
36				1.94	1.50	2.47
48				1.88	1.45	2.44
0					1.25	1.07
						1.75
36						5.38
						2.61
						4.28

Note: All strain values are in  $\text{in/in} \times 10^{-4}$

Table 6.6 The Shell Criteria - Limiting Subgrade Compressive Strain Values Corresponding to Different Load Applications

Load Repetitions	$10^5$	$10^6$	$10^7$	$10^8$
Axial Compressive strain in/in $\times 10^{-4}$	10.5	6.5	4.2	2.6

laboratory tests and from the BISAR output for the 18 and 30 kip loads. The  $10^8$  load applications for a 20 years service life was recommended by the FDOT for heavy traffic on an Interstate highway, (personal communications with Mr. William Lofroos, FDOT Pavement Design Engineer, Tallahassee, Florida, 1986). From Table 6.6 a value of  $2.61 \times 10^{-4}$  in/in strain is found to correspond to  $10^8$  load applications.

The comparisons show that for the laboratory strains, all tested subgrade heights would qualify as adequate, except for the A-5 which exceeds the criteria. For the strains output by BISAR for an 18 kip axle load, all heights qualify as adequate subgrade heights. For the BISAR 30 kips axle load, all heights qualify as adequate except for the A-2-4 at 30 inches and the A-5 at 36 inches. The BISAR program takes into account the relative stiffnesses of the pavement layers (i.e., the asphalt concrete, base, and stabilized subgrade). This leads to a reduction in the stresses and strains on top of the subgrade, compared to the Boussinesq solution and the laboratory testing results. The BISAR solution with the 30 kip axle load resulted in higher strains than in the laboratory testing, except in the A-5 soil. Based on the above, and to be on the safe side, the overloads have been considered in the comparisons to qualify certain subgrade heights above the water table. The comparisons were based on  $10^8$  load applications. Note that if  $10^7$

load applications were considered, the criteria strain would be  $4.2 \times 10^{-4}$  in/in (from Table 6.6) and all heights of subgrade fill, above the water table would qualify as adequate except the A-5. For  $10^6$  applications the A-5 with a height of 36 inches would also be adequate. The Shell Criteria strain for  $10^6$  load applications is  $6.5 \times 10^{-4}$  in/in. This strain level is about 30 percent higher than the strain for the A-5 soil under a 30 kip axle load. The 30 percent higher strain could be considered as a safety margin for this particular condition. Figure 6.16 shows the recommended subgrade clearance heights, above the water table, based on the overload of 30 kip for the project soils.

The above analysis agrees well with that made using the capillary fringe heights, and described in application number two of this chapter. The limitations of the Shell Company strain criteria, as stated in the literature review, should be closely followed for the above analysis to be valid.

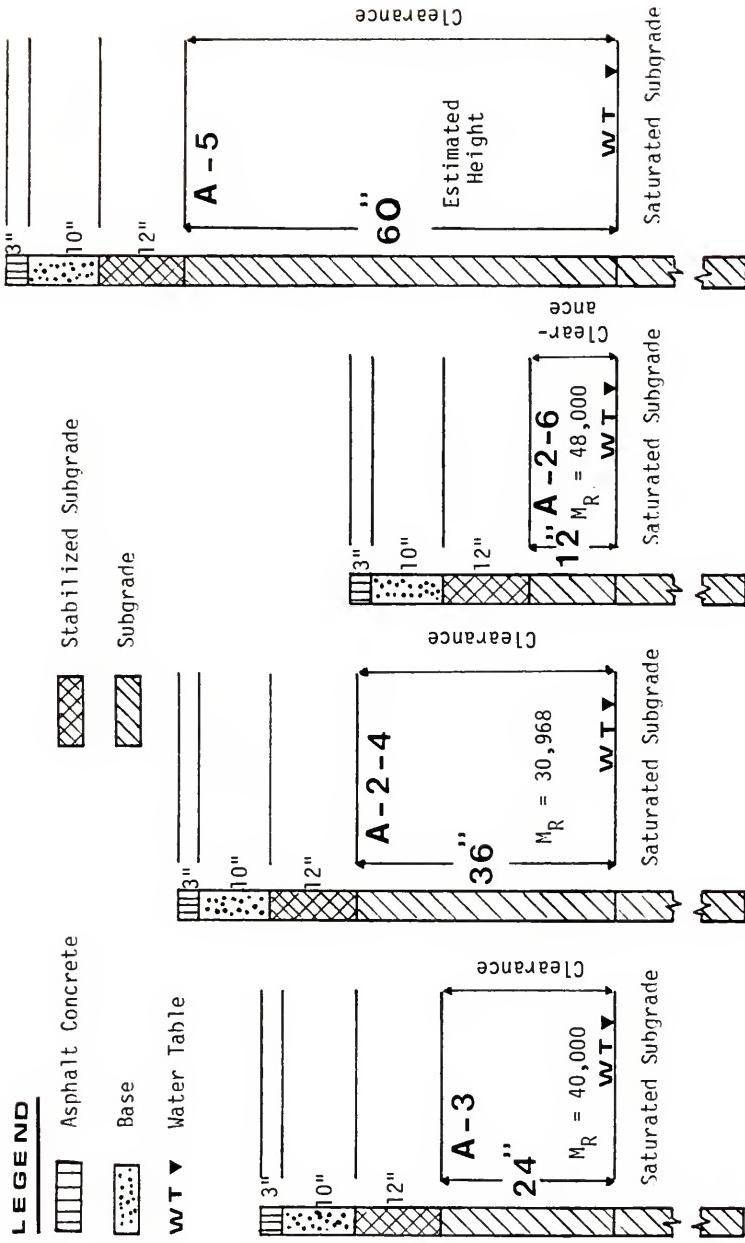


Figure 6.16 Recommended Subgrade Clearance Heights Based on a 30 kip Axle Load and  $10^8$  Load Applications for the Project Soils.

## CHAPTER VII CONCLUSIONS AND RECOMMENDATIONS

### 7.1 Conclusions

Based on the results of this investigation, the following conclusions are drawn:

1. Great emphasis should be placed on determining the designated highwater level. This was frequently mentioned in the questionnaire survey.
2. The soil-water retention curves determined using Tempe cells agreed well with the results from full scale column tests, as shown in Figure 6.6.
3. The compaction method had greater effect on the soil structure of cohesive soils than on granular soils. This was seen in the different soil-water characteristic curves, when specimens of the same soil were compacted by different methods (hand tools and the Rainhart compactor). This was very pronounced in the A-2-6 and A-5 soils, (see Figures 4.11 and 4.12, respectively).
4. Soil gradation, the percent passing the #200 sieve, and the mineralogy of the subgrade soil influence to a great extent the soil-water retention characteristics as shown in Figure 4.13.
5. The soil-water profiles that developed in the tested soil columns, after saturation then draining, showed water contents at the bottom of the columns near saturation and a continuous decrease above that level. This is quite different from the

placement conditions, which were at optimum water content throughout the soil columns' height. Figure 6.6 shows typical soil-water profiles.

6. The subgrade fill at different heights above the water table possesses different deformation characteristics. It is stronger the greater this height, as shown in Figures 6.7 and 6.8.
7. The deformation characteristics of the subgrade soil, i.e., the plot of accumulated axial permanent strain versus number of stress applications at different water retention conditions, can be represented by a power function of the form,

$$\epsilon_p^a = AN^B.$$

8. The A-2-6 soil tested in this research appears to be the best of the four project soils as a prospective subgrade fill.
9. The A-5 soil has a tendency to swell due to its montmorillonite content. It also possesses very high resilient strain.
10. During conditioning, in all soils, as the stress level increased, the resilient modulus decreased either consistently or to a minimum value, followed by a slight increase. Similar conclusions were reported by Morgan (1966) and Seed et al. (1962).
11. Based on both the soil-water characteristic capillary fringe and on an analysis using results from the repetitive load testing, the BISAR computer program and the Shell criteria, a subgrade fill of 24 inches is adequate for the A-3 soil, 36 inches for the A-2-4, and 12 inches for the A-2-6. If the A-5 soil must be used, a very careful analysis should be

- undertaken. A fill thickness in the range of 4 to 6 ft. is probably adequate.
12. The gradation of each soil can be related to its compaction characteristics. The well-graded A-2-6 soil showed the highest dry density at optimum. That in turn was reflected in the response of the soil during the repeated load testing where it experienced very small permanent strain. On the other hand, the A-2-4 soil experienced high permanent strain and high elastic strain due to its uniform gradation and poor drainability.
  13. Soil type is an important factor, and should be considered in the current guide lines for evaluating high water clearances in pavements. This is based on the variation in deformation characteristics found in the different soils.

### 7.2 Recommendations

The following recommendations are made for further study in this area:

1. Verify the effect of the engineering and physical properties of the subgrade on the deformation characteristic of the pavement system. This could be achieved by investigating two sections with approximately the same structure and traffic volume, one of which is performing well (less rut), the other poorly (excessive rut). The subgrade soils should be sampled and the engineering and physical properties determined. Emphasis should be placed on gradation, Atterberg limits, permeability, mineralogy, and the soil-water characteristics.

2. Tempe cells should be used as the primary mode of testing to obtain soil-water characteristic curves. Their advantages were outlined in Chapter 6. Column testing can only be justified when studying the interaction between different layers, such as subgrade and base or stabilized subgrade.

APPENDIX A

September 20, 1984

[address]

Dear [name]:

The enclosed questionnaire is part of a study sponsored by the Florida Department of Transportation (FDOT) to evaluate design highwater clearances for flexible highway pavements. The study is being conducted by the Geotechnical Group in the Department of Civil Engineering at the University of Florida.

Current policy of the FDOT dictates that the pavement base be located a specific height above a designated highwater level. The details are:

4-lane primary and Interstate	- 3 feet above 50 year highwater
2-lane primary	- 2 feet above 50 year highwater
secondary roads	- 1 foot above 25 year highwater

This policy has been applied uniformly statewide to satisfy two concerns:  
1) pavement performance and 2) construction operations.

The objective of the current study is to develop a more detailed set of guidelines for determining highwater clearances, taking into account, for example, soil type. It is hoped this will reduce the fill height requirement in some cases.

As an initial phase to the study, the attached questionnaire is being sent to all State Highway Departments. Please refer it to the most appropriate section within your department and return at your earliest convenience.

Your effort and participation is greatly appreciated.

Sincerely,

Mohamed Elfino, P.E.  
Ph.D. Candidate

ME/ltp

Enclosure

## UNIVERSITY OF FLORIDA - FLORIDA DOT, 1984

## QUESTIONNAIRE

Department Name: \_\_\_\_\_

Contact Person: \_\_\_\_\_

1. Does your department have similar guidelines?

yes \_\_\_\_\_ no \_\_\_\_\_

2. If the answer is NO, what do you have?

3. If the answer is yes, complete the following:

4-lane primary and Interstate - \_\_\_\_\_

2-lane primary - \_\_\_\_\_

Secondary roads - \_\_\_\_\_

4. How were your guidelines developed?

5. What is the procedure for implementing the guidelines?

6. Do you consider the soil type in your guidelines?

yes \_\_\_\_\_ no \_\_\_\_\_

7. If yes, how?

8. Do you consider the duration of the highwater level?

yes \_\_\_\_\_ no \_\_\_\_\_

9. If yes, how?
10. How do you determine the designated highwater level?
11. What is the most common material used for subgrade in your state?
- 1 -                    2 -
- 3 -                    4 -
- 5 -                    6 -
12. Do you think capillary rise would deteriorate the base and subgrade?
- yes \_\_\_\_\_ no \_\_\_\_\_
13. Have you had any problems caused by capillary rise?
- yes \_\_\_\_\_ no \_\_\_\_\_
14. If yes, explain with an example.
15. How do you evaluate pavement deformation-type of tests?
16. What criteria are used in your department to evaluate pavement deformation? (i.e., Shell Oil Co., Chevron Oil Co., Asphalt Institute)

17. What type of measures are taken to eliminate the capillary rise problem?
18. Have you studied or are you aware of any studies on highwater clearances?  
yes \_\_\_\_\_ no \_\_\_\_\_
19. If yes, give details of dates, reports, the availability, etc.
20. Other comments or suggestions:

APPENDIX B

Table B.1 Soil-Water Retention Test Results

Soil Type: A-3

$\gamma_d$ : 102.5 pcf

Method of Compaction: Standard Proctor

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	36.73	22.36
20.0	34.82	21.20
30.0	33.30	20.27
45.0	17.46	10.63
60.0	12.32	7.50
80.0	9.30	5.66
100.0	8.05	4.90
150.0	6.90	4.20
200.0	6.54	3.98
345.0	5.85	3.56

Table B.2 Soil-Water Retention Test Results

Soil Type: A-2-4

 $\gamma_d$ : 106.73 pcf

Method of Compaction: Standard Proctor

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	29.83	17.44
20.0	29.27	17.11
30.0	29.18	17.06
45.0	28.56	16.70
60.0	27.23	15.92
80.0	19.46	11.38
100.0	17.10	10.00
150.0	14.37	8.40
200.0	13.39	7.83
345.0	11.65	6.81

Table B.3 Soil-Water Retention Test Results

Soil Type: A-2-6

 $\gamma_d$ : 119.95 pcf

Method of Compaction: Standard Proctor

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	27.76	14.44
20.0	26.80	13.94
30.0	26.68	13.88
45.0	25.93	13.49
60.0	25.82	13.43
80.0	25.49	13.26
100.0	25.20	13.11
150.0	24.76	12.88
200.0	24.49	12.74
345.0	23.68	12.32

Table B.4 Soil-Water Retention Test Results

Soil Type: A-5

 $\gamma_d$ : 70.95pcf

Method of Compaction: Standard Proctor

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	56.13	49.37
20.0	56.13	49.37
30.0	56.08	49.32
45.0	55.40	48.72
60.0	55.07	48.43
80.0	54.67	48.08
100.0	53.63	47.17
150.0	53.00	46.61
200.0	52.85	46.48
345.0	49.03	43.12

Table B.5 Soil-Water Retention Test Results

Soil Type: A-3

 $\gamma_d$ : 101.10 pcf

Method of Compaction: Hand Tools

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	32.88	20.29
20.0	30.02	18.53
30.0	21.09	13.02
45.0	14.34	8.85
60.0	10.63	6.59
80.0	8.32	5.14
100.0	7.59	4.69
150.0	6.63	4.09
200.0	6.49	4.00
345.0	5.67	3.50

Table B.6 Soil-Water Retention Test Results

Soil Type: A-2-4

 $\gamma_d$ : 104.83 pcf

Method of Compaction: Hand Tools

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	29.58	17.56
20.0	28.94	17.18
30.0	28.86	17.13
45.0	28.19	16.73
60.0	20.45	12.14
80.0	16.11	9.56
100.0	14.42	8.56
150.0	12.68	7.52
200.0	12.01	7.13
345.0	10.44	6.19

Table B.7 Soil-Water Retention Test Results

Soil Type: A-2-6

 $\gamma_d$ : 121.01 pcf

Method of Compaction: Hand Tools

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	22.26	11.49
20.0	22.24	11.47
30.0	22.21	11.46
45.0	22.12	11.41
60.0	22.12	11.41
80.0	22.06	11.38
100.0	22.06	11.38
150.0	22.00	11.35
200.0	21.89	11.29
345.0	21.60	11.14

Table B.8 Soil-Water Retention Test Results

Soil Type: A-5

 $\gamma_d$ : 69.26 pcf

Method of Compaction: Hand Tools

Pressure cm Water	Water Content Percentage	
	Volumetric	Gravimetric
3.5	52.99	47.68
20.0	52.46	47.21
30.0	51.91	46.71
45.0	51.76	46.58
60.0	51.47	46.32
80.0	51.36	46.22
100.0	51.15	46.03
150.0	51.10	45.98
200.0	50.95	45.85
345.0	---	---

APPENDIX C

Table C.1 Repetitive Load Testing Results, A-3 Soil, w = 14.8 Percent

Soil Type: A-3 Specimen No.: 3-0pt.  
Axial Load: 8.0 psi  $\gamma_d$ : 103.1pcf  
Confining Pressure: 2.0 psi Water Content: 14.8 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	1.1	1.38	43,636	0.7	1.44	1.04
10	2.8	1.44	41,739	2.9	1.12	0.78
100	10.1	1.88	32,000	11.1	1.31	0.70
200	17.9	1.81	33,103	19.7	1.37	0.76
400	25.4	2.25	26,667	26.1	1.31	0.58
600	31.6	1.94	30,968	31.2	1.25	0.64
800	34.9	2.25	26,667	34.4	1.31	0.58
1000	37.2	2.19	27,429	35.4	1.31	0.60
2000	48.7	2.44	24,615	44.9	1.25	0.51
3000	53.4	1.75	34,286	47.6	1.19	0.68
4000	57.8	1.88	32,000	51.4	1.19	0.63
5000	60.7	1.63	36,923	52.9	0.94	0.58
6000	63.4	1.75	34,286	55.0	1.00	0.57
7000	66.7	1.63	36,923	58.2	1.00	0.61
8000	68.1	1.75	34,286	58.9	1.00	0.57
9000	69.3	1.75	34,286	59.3	1.06	0.61
10000	70.5	1.69	35,556	60.0	0.87	0.52

Table C.2 Repetitive Load Testing Results, A-2-4 Soil, w = 11.8 Percent

Soil Type: A-2-4

Specimen No.: 1-Opt.

Axial Load: 8.0 psi

 $\gamma_d$ : 107.2 pcf

Confining Pressure: 2.0 psi

Water Content: 11.8 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.13	2.53	23,700	0.25	3.16	1.25
10	1.01	2.78	21,545	2.15	3.28	1.18
100	6.84	2.91	20,609	11.90	3.41	1.17
200	12.30	2.72	22,047	20.10	2.90	1.07
400	21.30	2.78	21,545	31.90	3.16	1.14
600	26.70	3.04	19,750	38.00	2.71	0.89
800	31.80	2.72	22,047	43.10	2.71	1.00
1000	37.70	2.41	24,947	47.70	2.53	1.05
2000	49.40	2.53	23,700	54.20	2.78	1.10
3000	54.30	2.66	22,571	57.70	2.78	1.05
4000	58.90	2.53	23,700	61.50	2.65	1.05
5000	62.00	2.53	23,700	64.30	2.71	1.07
6000	64.60	2.53	23,700	66.30	2.65	1.05
7000	67.10	2.53	23,700	68.40	2.78	1.10
8000	68.40	2.85	21,067	69.80	2.65	0.93
9000	69.90	2.85	21,067	70.70	2.65	0.93
10000	72.00	2.66	22,571	72.30	2.65	1.00

Table C.3 Repetitive Load Testing Results, A-2-6 Soil, w = 11.6 Percent

Soil Type: A-2-6 Specimen No.: 6-Opt.  
 Axial Load: 8.0 psi  $\gamma_d$ : 120.3pcf  
 Confining Pressure: 2.0 psi Water Content: 11.6 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.30	2.13	28,235	0.1	0.63	0.30
10	0.40	1.94	30,968	0.1	0.63	0.32
100	0.60	1.88	32,000	0.1	0.63	0.33
200	0.80	1.75	34,286	0.1	0.56	0.32
400	0.90	1.81	33,103	0.1	0.56	0.31
600	1.10	1.77	33,898	0.1	0.50	0.32
800	1.40	1.50	40,000	0.1	0.25	0.17
1000	1.60	1.56	38,400	0.1	0.25	0.16
2000	2.30	1.25	48,000	0.1	0.25	0.20
3000	2.75	1.13	53,333	0.1	0.13	0.11
4000	3.00	1.13	53,333	0.1	0.13	0.11
5000	3.12	1.13	53,333	0.1	0.13	0.11
6000	3.38	1.00	60,000	0.1	0.13	0.13
7000	3.60	1.00	60,000	0.1	0.13	0.13
8000	3.60	1.00	60,000	0.1	0.13	0.13
9000	3.60	1.00	60,000	0.1	0.13	0.13
10000	3.60	1.00	60,000	0.1	0.13	0.13

Table C.4 Repetitive Load Testing Results, A-5 Soil, w = 37.84 Percent

Soil Type: A-5

Specimen No.: 3-Opt.

Axial Load: 8.0 psi

 $\gamma_d$ : 72.0 pcf

Confining Pressure: 2.0 psi

Water Content: 37.84 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.4	5.63	10,667	0.0	2.52	0.45
10	0.4	5.75	10,435	0.0	2.52	0.44
100	1.0	5.56	10,787	0.1	2.52	0.45
200	1.0	5.56	10,787	0.0	2.52	0.45
400	1.3	5.88	10,213	0.0	2.52	0.43
600	2.2	5.44	11,034	0.0	2.52	0.42
800	2.8	5.63	10,667	0.3	2.52	0.45
1000	3.4	5.50	10,909	0.6	2.52	0.46
2000	4.7	5.13	11,707	0.8	2.52	0.49
3000	4.9	5.25	11,429	0.3	2.52	0.48
4000	6.2	5.25	11,429	0.3	2.52	0.48
5000	6.2	5.00	12,000	0.3	2.39	0.48
6000	6.4	5.00	12,000	0.3	2.39	0.48
7000	6.6	5.00	12,000	0.3	2.39	0.48
8000	6.6	5.00	12,000	0.3	2.27	0.45
9000	6.6	5.00	12,000	0.3	2.14	0.43
10000	6.6	5.00	12,000	0.3	2.27	0.45

Table C.5 Repetitive Load Testing Results, A-3 Soil, w = 13.25 Percent

Soil Type: A-3 Specimen No.: 6-15  
 Axial Load: 8.0 psi  $\gamma_d$ : 103.1pcf  
 Confining Pressure: 2.0 psi Water Content: 13.25 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.8	1.75	34,286	0.5	0.63	0.36
10	1.8	1.69	35,556	1.7	0.69	0.41
100	5.8	1.69	35,556	6.6	0.63	0.35
200	8.8	1.81	33,103	10.3	0.63	0.37
400	12.3	1.75	34,286	14.5	0.57	0.33
600	15.3	1.81	33,103	18.4	0.50	0.28
800	17.4	1.88	32,000	20.7	0.57	0.30
1000	19.1	1.88	32,000	23.0	0.57	0.30
2000	22.8	2.13	28,235	25.7	0.50	0.23
3000	25.5	1.88	32,000	27.9	0.50	0.27
4000	27.2	2.13	28,235	29.7	0.76	0.36
5000	28.1	2.19	27,429	30.1	0.69	0.32
6000	29.4	2.13	28,935	30.7	0.63	0.30
7000	30.5	1.94	30,968	31.5	0.63	0.32
8000	31.2	2.19	27,429	31.8	0.63	0.29
9000	32.0	2.13	28,235	32.7	0.63	0.30
10000	32.6	2.13	28,235	33.0	0.63	0.30

Table C.6 Repetitive Load Testing Results, A-3 Soil, w = 10.53 Percent

Soil Type: A-3 Specimen No.: 5-18  
Axial Load: 8.0 psi  $\gamma_d$ : 103.7 pcf  
Confining Pressure: 2.0 psi Water Content: 10.53 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.9	1.06	56,471	0.3	0.50	0.25
10	1.9	1.31	45,714	1.1	0.69	0.53
100	5.0	1.38	43,636	3.9	0.50	0.36
200	6.1	1.63	36,923	4.8	0.56	0.34
400	8.3	1.50	40,000	6.6	0.56	0.37
600	10.4	1.63	36,923	8.9	0.38	0.23
800	11.5	1.63	36,923	10.2	0.25	0.15
1000	12.7	1.69	35,553	11.4	0.25	0.14
2000	16.3	1.69	35,556	15.0	0.25	0.14
3000	18.4	1.81	33,103	16.3	0.19	0.11
4000	19.4	1.75	34,286	16.6	0.25	0.14
5000	20.5	1.69	35,556	17.4	0.25	0.14
6000	21.7	1.56	38,400	18.13	0.13	0.08
7000	22.8	1.69	35,556	19.3	0.25	0.14
8000	23.6	1.56	38,400	19.9	0.25	0.16
9000	24.4	1.75	34,286	20.6	0.25	0.14
10000	25.8	1.75	34,286	22.8	0.13	0.07

Table C.7 Repetitive Load Testing Results, A-3 Soil, w = 8.10 Percent

Soil Type: A-3 Specimen No.: 4-24  
 Axial Load: 8.0 psi  $\gamma_d$ : 102.9 pcf  
 Confining Pressure: 2.0 psi Water Content: 8.10 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.8	1.38	43,636	0.2	0.19	0.14
10	1.4	1.44	41,739	0.6	0.38	0.26
100	2.7	1.56	38,400	2.6	0.25	0.16
200	3.2	1.56	38,400	3.5	0.19	0.12
400	4.3	1.50	40,000	5.1	0.19	0.13
600	5.1	1.50	40,000	6.5	0.19	0.13
800	5.5	1.63	36,923	7.2	0.19	0.12
1000	5.9	1.56	38,400	7.7	0.19	0.13
2000	7.0	1.63	36,923	9.2	0.13	0.08
3000	8.0	1.50	40,000	10.5	0.13	0.09
4000	8.6	1.63	36,923	11.5	0.13	0.08
5000	9.0	1.63	36,923	11.9	0.13	0.08
6000	9.9	1.31	45,714	12.4	0.06	0.05
7000	10.5	1.38	43,636	13.0	0.06	0.04
8000	11.4	1.38	43,636	14.8	0.06	0.04
9000	12.0	1.38	43,636	15.6	0.06	0.04
10000	12.6	1.50	40,000	16.8	0.06	0.04

Table C.8 Repetitive Load Testing Results, A-2-4 Soil, w = 11.70 Percent

Soil Type: A-2-4 Specimen No.: 1-30  
 Axial Load: 8.0 psi  $\gamma_d$ : 107.7 pcf  
 Confining Pressure: 2.0 psi Water Content: 11.70 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	4.40	1.59	37,723	6.20	1.37	0.86
10	9.40	2.29	26,197	13.80	2.30	1.00
100	31.90	2.29	26,197	50.70	1.87	0.82
200	42.00	2.42	24,818	67.60	1.38	0.57
400	54.70	2.54	23,577	91.50	1.18	0.46
600	68.50	2.10	28,578	117.80	0.87	0.41
800	75.60	1.91	31,436	129.40	0.87	0.46
1000	79.00	2.61	27,738	134.40	1.12	0.43
2000	89.30	2.10	28,578	155.20	0.75	0.36
3000	107.30	1.86	32,321	185.00	1.05	0.56
4000	113.00	1.98	30,236	195.00	1.05	0.53
5000	117.00	1.98	30,236	202.30	0.87	0.44
6000	118.60	2.05	29,291	205.20	0.99	0.48
7000	119.90	2.24	26,781	207.60	1.05	0.47
8000	121.60	2.05	29,291	210.40	1.05	0.51
9000	123.00	2.18	27,568	213.70	1.18	0.54
10000	123.40	2.43	24,666	215.00	1.05	0.43

Table C.9 Repetitive Load Testing Results, A-2-4 Soil, w = 10.50 Percent

Soil Type: A-2-4 Specimen No.: 2-35  
Axial Load: 8.0 psi  $\gamma_d$ : 107.4 pcf  
Confining Pressure: 2.0 psi Water Content: 10.50 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	2.40	1.88	32,000	5.00	0.19	0.10
10	4.40	1.81	33,103	9.30	2.01	1.11
100	10.10	2.13	28,235	20.60	2.45	1.15
200	13.60	2.00	30,000	27.90	2.39	1.19
400	18.10	2.25	26,667	37.20	2.26	1.01
600	22.80	2.13	28,235	46.9	2.39	1.12
800	26.10	2.13	28,235	54.00	1.76	0.83
1000	29.40	2.13	28,235	61.40	1.70	0.80
2000	36.90	2.25	26,667	77.90	1.51	0.67
3000	40.80	2.19	27,429	85.90	1.26	0.58
4000	43.30	2.31	25,946	91.10	1.57	0.68
5000	46.40	2.19	27,429	97.00	1.19	0.55
6000	48.60	1.69	35,556	100.60	1.19	0.71
7000	51.60	1.69	35,556	109.10	1.19	0.71
8000	53.20	1.81	33,103	112.30	1.26	0.69
9000	54.3	1.81	33,103	114.30	1.32	0.73
10000	55.10	1.94	30,968	116.00	1.38	0.71

Table C.10 Repetitive Load Testing Results, A-2-4 Soil, w = 8.7 Percent

Soil Type: A-2-4

Specimen No.: 3-43

Axial Load: 8.0 psi

 $\gamma_d$ : 108.5pcf

Confining Pressure: 2.0 psi

Water Content: 8.7 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.13	2.19	27,429	0.06	1.32	0.60
10	0.25	2.31	25,946	0.13	1.38	0.60
100	1.31	2.31	25,946	1.25	1.24	0.54
200	2.00	2.19	27,429	1.25	1.25	0.57
400	2.63	2.50	24,000	1.38	1.38	0.55
600	3.31	2.06	29,091	1.88	1.25	0.61
800	4.31	2.00	30,000	2.57	1.32	0.66
1000	4.81	2.00	30,000	2.82	1.25	0.63
2000	5.75	1.94	30,968	3.20	1.19	0.61
3000	6.88	1.94	30,968	3.76	1.25	0.64
4000	7.13	1.88	32,000	3.76	1.25	0.66
5000	8.50	1.88	32,000	4.51	1.25	0.66
6000	8.94	2.00	30,000	4.26	1.38	0.69
7000	9.13	2.31	25,946	4.01	1.32	0.57
8000	9.19	2.50	24,000	3.76	1.38	0.55
9000	9.69	2.13	28,235	3.76	1.32	0.62
10000	10.13	1.88	32,000	3.76	1.38	0.55

Table C.11 Repetitive Load Testing Results, A-2-6 Soil,  
 $w = 12.20$  Percent

Soil Type: A-2-6

Specimen No.: 1-Sat.

Axial Load: 8.0 psi

$\gamma_d$ : 119.5 pcf

Confining Pressure: 2.0 psi

Water Content: 12.20 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.01	1.24	48,485	0.01	0.62	0.50
10	0.01	1.24	48,485	0.01	0.62	0.50
100	0.13	1.25	48,000	0.01	0.74	0.59
200	0.19	1.31	45,801	0.13	1.01	0.81
400	0.38	1.31	45,801	0.13	1.01	0.81
600	0.44	1.31	45,801	0.19	1.01	0.81
800	0.50	1.38	43,478	0.25	1.01	0.78
1000	0.63	1.38	43,478	0.38	1.01	0.73
2000	1.13	1.31	45,801	0.76	1.01	0.78
3000	1.56	1.25	48,000	1.26	0.88	0.71
4000	1.63	1.25	48,000	1.26	0.88	0.71
5000	1.75	1.25	48,000	1.26	0.88	0.71
6000	1.88	1.25	48,000	1.26	0.88	0.71
7000	2.00	1.25	48,000	1.26	0.88	0.71
8000	2.00	1.25	48,000	1.38	0.87	0.70
9000	2.00	1.25	48,000	1.38	0.87	0.70
10000	2.00	1.25	48,000	1.38	0.87	0.70

Table C.12 Repetitive Load Testing Results, A-5 Soil, w = 44.4 Percent

Soil Type: A-5

Specimen No.: 1-36

Axial Load: 8.0 psi

 $\gamma_d$ : 72.1 pcf

Confining Pressure: 2.0 psi

Water Content: 44.4 percent

Cycle	Axial Strain $\times 10^{-4}$ in/in		Resil Mod psi	Radial Strain $\times 10^{-4}$ in/in		Resil Poisson's Ratio
	perm	resil		perm	resil	
1	0.19	5.81	10,327	0.13	6.37	1.10
10	0.38	5.87	10,221	0.19	7.87	1.34
100	0.94	5.94	10,101	0.81	8.13	1.37
200	1.44	5.94	10,101	1.13	7.87	1.32
400	2.06	5.94	10,101	1.63	7.87	1.32
600	2.38	5.93	10,118	1.94	7.87	1.33
800	2.69	5.93	10,118	2.25	7.81	1.32
1000	2.88	5.93	10,118	2.63	7.87	1.33
2000	4.69	5.37	11,173	4.13	7.37	1.37
3000	5.56	5.38	11,152	4.69	7.44	1.38
4000	5.81	5.38	11,152	4.94	7.44	1.38
5000	6.06	5.38	11,152	5.31	7.44	1.38
6000	6.06	5.38	11,152	5.31	7.44	1.38
7000	6.06	5.38	11,152	5.31	7.44	1.38
8000	6.06	5.38	11,152	5.31	7.44	1.38
9000	6.06	5.38	11,152	5.31	7.44	1.38
10000	6.06	5.38	11,152	5.31	7.44	1.38

Table C.13 Repetitive Load Conditioning Results, A-3 Soil

Soil Type: A-3			Specimen No.: 3-0pt.		
$\gamma_d$ : 103.1 pcf			Water Content: 14.8 percent		
$\sigma_1/\sigma_3 = 2$			$\sigma_1/\sigma_3 = 3$		
Axial Strain $\times 10^{-4}$ in/in			Axial Strain $\times 10^{-4}$ in/in		
Cycle	perm resil	Resil Mod psi	perm resil	Resil Mod psi	perm resil
1	0.375	-- <sup>a</sup>	-b	1.625	-- <sup>a</sup>
10	0.500	--	--	3.875	--
20	0.625	--	--	5.250	--
40	0.750	--	--	6.125	--
60	0.875	--	--	6.750	--
80	0.875	--	--	7.250	--
100	0.875	--	--	7.500	--
200	1.125	--	--	9.125	--

## Notes:

a The resilient strain values were too small to detect with the available equipment.

b Resilient modulus values were not reported based on note a.

Table C.14 Repetitive Load Conditioning Results, A-2-4 Soil

Soil Type: A-2-4  
 Specimen No.: 1-0pt.  
 $\gamma_d$ : 107.2 pcf  
 Water Content: 11.8 percent

Cycle	$\sigma_1/\sigma_3 = 2$		$\sigma_1/\sigma_3 = 3$		$\sigma_1/\sigma_3 = 4$	
	Axial Strain $\times 10^{-4}$ in/in		Axial Strain $\times 10^{-4}$ in/in		Axial Strain $\times 10^{-4}$ in/in	
	perm	resil	Mod psi	perm	resil psi	Mod psi
1	0.438	0.437	45,766	2.500	1.937	37,774
10	0.563	0.437	45,766	4.063	1.937	37,774
20	0.625	0.437	45,766	4.813	1.937	37,774
40	0.750	0.437	45,766	5.875	1.875	21,333
60	0.813	0.437	45,766	6.375	2.000	20,000
80	0.813	0.437	45,766	7.063	1.875	21,333
100	0.875	0.437	45,766	7.375	2.000	20,000
200	1.063	0.437	45,766	9.750	1.750	22,857

Table C.15 Repetitive Load Conditioning Results, A-2-5 Soil

Soil Type:	A-2-6	Specimen No.:	-opt.
$\gamma_d$	120.3 ppcf	Water Content:	11.7 percent

$\sigma_1/\sigma_3 = 2$		$\sigma_1/\sigma_3 = 3$		$\sigma_1/\sigma_3 = 4$	
Cycle	Axial Strain $\times 10^{-4}$ in/in	Axial Strain $\times 10^{-4}$ in/in		Axial Strain $\times 10^{-4}$ in/in	
		Resil Mod psi	perm	Resil Mod psi	perm
1	0.125	-- <sup>a</sup>	-- <sup>b</sup>	0.688	1.187
10	0.188	--	--	0.938	1.125
20	0.213	--	--	1.063	1.125
40	0.250	--	--	1.125	1.313
60	0.313	--	--	1.188	1.313
80	0.313	--	--	1.313	1.250
100	0.313	--	--	1.313	1.250
200	0.313	--	--	1.313	1.250

Notes:

a The resilient strain values were too small to be detected by the available equipment.

Resilient modulus values were not reported based on note a.

Table C.16 Repetitive Load Conditioning Results, A-5 Soil

Soil Type: A-5			Specimen No.: 3-0pt.		
$\gamma_d$ : 72.0 pcf			Water Content: 37.84 percent		
$\sigma_1/\sigma_3 = 2$			$\sigma_1/\sigma_3 = 3$		
Cycle	Axial Strain $\times 10^{-4}$ in/in	Axial Strain $\times 10^{-4}$ in/in	Axial Strain $\times 10^{-4}$ in/in	Axial Strain $\times 10^{-4}$ in/in	
	perm resil psi	perm resil psi	perm resil psi	perm resil psi	perm resil psi
1	0.375	0.500	40,000	3.063	2,500
10	0.813	0.750	26,667	3.625	3,000
20	1.063	0.750	26,667	4.188	2,875
40	1.375	0.625	32,000	4.438	3,000
60	1.625	0.625	32,000	4.688	3,187
80	1.750	0.625	32,000	4.938	3,187
100	1.875	0.625	32,000	5.000	3,125
200	2.188	0.625	32,000	5.063	3,125

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## BIOGRAPHICAL SKETCH

Mohamed K. Elfino was born on July 5, 1945, in Alexandria, Egypt. After graduation from high school in 1962, he attended Alexandria University, Egypt, and graduated with a Bachelor of Science degree in agricultural engineering in June 1967.

Between 1967 and 1970, he worked for the General Authority of Agrarian Reform, Alexandria, Egypt, as an agricultural engineer.

In January 1971, he moved to the United States of America, and in November 1972 he joined Ring Power Corporation, Jacksonville, Florida, as a preventive maintenance program coordinator.

In January 1978, he enrolled in the Department of Agricultural Engineering, University of Florida, as a graduate research assistant and graduated with the degree of Master of Engineering in agricultural engineering in June 1980.

Between January 1980 and January 1985, he worked full time, as a project engineer for the Agricultural Engineering Department, University of Florida. Also during this period, he pursued work toward his Ph.D. in civil engineering on a part-time basis.

From January 1985 until present, he devoted his entire time to pursue his doctoral studies.

He is a registered Professional Engineer in the State of Florida, and the author and co-author of fourteen technical papers. He is a member of the American Society of Civil Engineers, the American Society

of Agricultural Engineers, a member of Alpha Epsilon Honor Society of Agricultural Engineers.

Mohamed Elfino is married to the former Nariman M. Elhawary of Alexandria, Egypt. They have a 9 year-old daughter, Nancy.

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Associate Professor of Civil Engineering

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Frank C. Townsend  
Professor of Civil Engineering

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Walter H. Zimpfer  
Associate Professor of Civil Engineering

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James L. Eades  
Associate Professor of Geology

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Luther C. Hammond  
Luther C. Hammond  
Professor of Soil Science

This dissertation was submitted to the Graduate Faculty of the College of Engineering and to the Graduate School and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy.

May 1986

W. G. Shafer  
Dean, College of Engineering

Dean, Graduate School

UNIVERSITY OF FLORIDA



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